WATER & SEWAGE WORKS

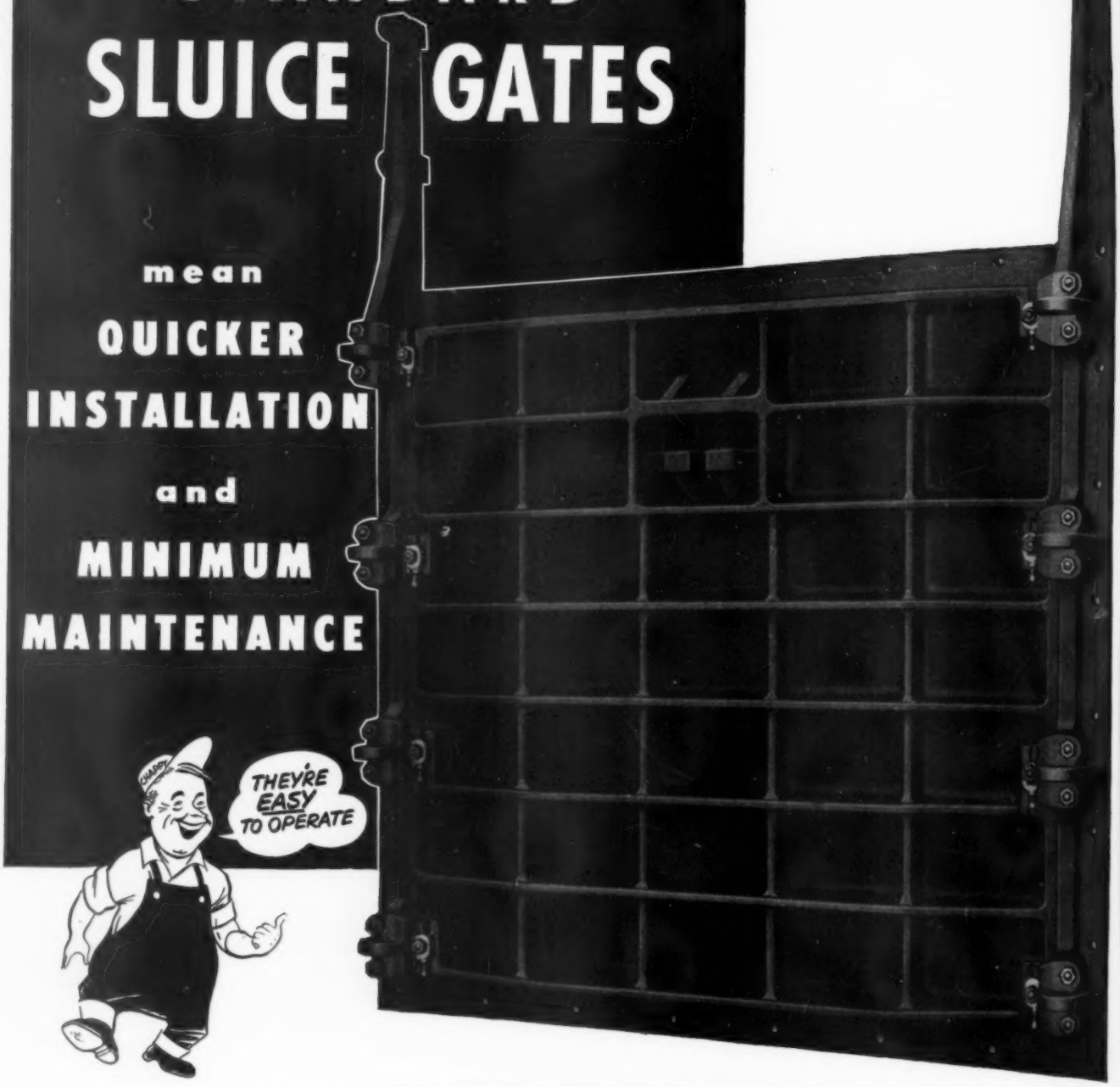
CHAPMAN
STANDARD
SLUICE GATES

mean

QUICKER INSTALLATION

and

MINIMUM MAINTENANCE



Chapman Standard Sluice Gates are easy to install . . . easy to maintain . . . easy to operate. They are standardized to meet all the usual conditions, and quicker installation and less maintenance are assured because all stems and couplings are completely interchangeable . . . no match-marking is necessary. Repair and replacement parts can be speedily installed in the field without alterations.

What's more, you can get Chapman Sluice Gates with any type of operating control—manual, hydraulic cylinder, or motor unit. The motor unit is completely enclosed and weatherproof; it is wired at the factory, and installation is simply a matter of connecting the power leads. Write *today* for complete information.

THE CHAPMAN VALVE MANUFACTURING CO.

INDIAN ORCHARD, MASSACHUSETTS

WATER AND SEWAGE WORKS. Reg. U. S. Patent Off., April. 1948. Volume 95. Number 4. Published monthly by Gillette Publishing Co. at 404 North Wesley Ave., Mount Morris, Ill. Per copy, 25c; except *this issue*, 75 cents. Subscription rate: United States, $2.00 per year; Canada, $2.00 per year; Foreign, $3.00 per year. Entered as second-class matter March 4, 1948, at the post office at Mount Morris, Illinois, under the Act of March 3, 1879. Executive and Business offices, 22 W. Maple St., Chicago 10, Illinois. Editorial office, 155 E. 44th St., New York 17, N. Y.

WHAT'S THE BEST METER FOR THIS SERVICE ?

For a better buy .... SELECT FROM THE ONLY COMPLETE LINE

Keeping a variety of water services metered at peak efficiency can be a big task. But it needn't be—with the help of the complete Pittsburgh-Empire line. When in doubt about the best type of meter for any service, rely on the *impartial* advice of your Pittsburgh-Empire representative. He handles *all* the commercial varieties of meters, hence has no axe to grind. These men have second-to-none records for applying the right meter to all types of water measurement jobs.

For unprejudiced guidance, for real measurement engineering service, and for a complete line of meters from which to choose you have only to go to one source—Pittsburgh-Empire. And you can be sure to receive top quality and the best in measurement value from any Pittsburgh-Empire meter you buy.

We cordially invite

all members of the A.W.W.A. to meet their friends during the annual convention at our headquarters. Hotel Shelburne, Atlantic City, N. J.

better buy

PITTSBURGH-EMPIRE WATER METERS

PITTSBURGH EQUITABLE METER DIVISION
Rockwell Manufacturing Company, Pittsburgh 8, Pa.

Atlanta Boston Chicago Houston Kansas City Los Angeles
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YOUR UNRESTRICTED CHOICE OF SIZE AND TYPE

Pittsburgh Arctic Disc Meter

Pittsburgh Tropic Disc Meter

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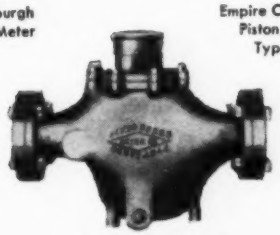
Empire Oscillating Piston Meter Type 12

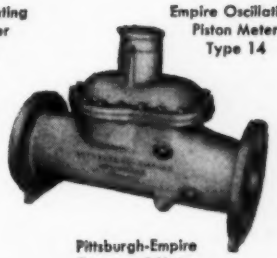
Empire Oscillating Piston Meter Type 14

Pittsburgh Disc Meter, Large Capacity Type

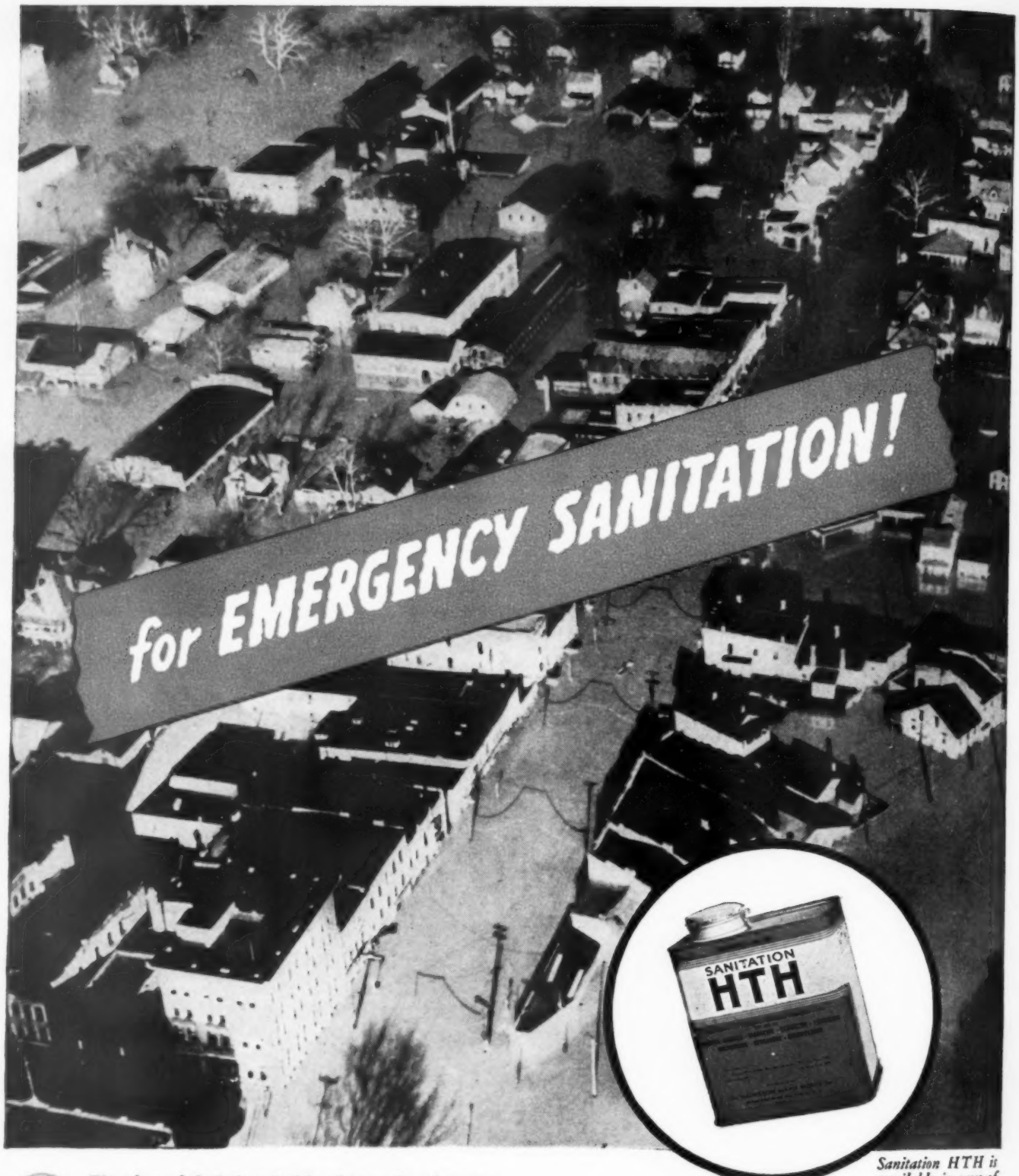
Empire Oscillating Piston Meter Type 16



Pittsburgh Eureka "S" Meter, Current Type



Pittsburgh-Empire Compound Meter Single Register Type



Floods and their inevitable aftermath—the menace of epidemic water-borne disease—call for quick and effective action. Countless communities stricken by flood waters know how Mathieson's Sanitation HTH affords prompt chlorination of polluted water supplies—prevents the scourge of disease and makes possible early resumption of safe water service.

Flood threatened communities should maintain adequate stocks of Sanitation HTH to secure the protection its 70% available chlorine provides in meeting flood and other emergency conditions. Check your stock now—and write or wire for an additional supply of Sanitation HTH if required.

Sanitation HTH is available in cases of nine 5-lb. cans and in 100-lb. drums.

Mathieson
CHEMICALS

THE MATHIESON ALKALI WORKS (INC.)
60 East 42nd Street, New York 17, N.Y.

SANITATION HTH . . . LIQUID CHLORINE . . . PH-PLUS . . . FUSED ALKALI
CAUSTIC SODA . . . SODA ASH . . . BICARBONATE OF SODA
CHLORINE DIOXIDE . . . AMMONIA, ANHYDROUS & AQUA . . . DRY ICE
CARBONIC GAS . . . SODIUM CHLORITE PRODUCTS . . . SODIUM METHYLATE

WATER & SEWAGE WORKS

E. S. GILLETTE, Publisher

L. H. ENSLOW, Editor

COMING!

THIS MONTH'S COVER

Spillway and Outflow of Wanaque Reservoir North Jersey Distr. Water Supply Commission.

See article in this issue by Chas. A. Capen, Chief Engineer of the No. Jersey District System.

Our Relations Are Always With Us—

So says the writer of this open letter to Doc Symons in answer to his query, "How Are Your Relations—Public, That Is?" Done in the highly readable style of personal correspondence, this "letter" tells how the art of public relations is practiced by

JOHN E. KLEINHENZ, Publicity,
Indianapolis (Ind.) Water Co.

Sludge Digestion Practices—

Is the subject of a series of articles by a keen student and observer of sewage treatment practices. This series, now running, compares the various methods of sludge digestion currently practiced, with suggestions and recommendations born of observations, coupled with experience of practical plant operators. The author, who needs no introduction to our older readers is—

LeROY Van KLEECK, San. Engr.,
Conn. State Dept. of Health, Hartford

Chemistry of Water Treatment

Is of particular interest to water purification engineers and chemists, but this series of papers on coagulation and water softening will be read with profit by any person in the field of water supply. These fundamental articles, the first of which appears in this issue, have been prepared from lectures given before a short school by—

DR. A. P. BLACK,
Prof. of Chemistry, Univ. of Florida

New Heating System For Sludge Digestion

Describes the new sludge heating system being installed in the Philadelphia Sewage Treatment Works, now under construction. The method and equipment is novel, and is claimed to be flexible and economical, in that digester gas is burned under pressure within a combustion unit submerged in the raw sludge en-route to the digesters. The authors are—

E. R. SCOFIELD AND M. B. TARK, Design Engrs.
Dept. of Public Works, Philadelphia, Pa.

Determining Residual Chlorine In Chlorinated Sewages

Constitutes the first of a series of articles which will compare available methods of determining residual chlorine in sewages, and attempt to explain the cause of the marked discrepancy between the methods, the objective being that of choosing the most practical and most accurate method for conditions commonly experienced in sewage chlorination. This contribution comes from the Research and Development Laboratories of Wallace and Tiernan Co.—

MESSRS. MARKS, JOINER AND STRANDSKOV

Delayed—

One issue—will be the report covering the March meeting of N.E.W.W.A. prepared for this issue.

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WATER AND SEWAGE WORKS is a monthly publication. Single Copies, 25c; Subscription Rates, U. S. and Canada, $2 per year; Foreign, $3.

THE CUSTOMERS TELL US

This is a "man bites dog" version of what we have been saying in our advertisements for these many years — namely, that the great majority of the *original* cast iron water mains, laid in this country from the earliest days, are still in service. Now our customers give *us* the facts—facts secured *by* users, *from* users, *for* users.

The chart shown on the opposite page is based on a published report of the American Water Works Association of the results of a survey of the "Survival and Retirement of Water Works Facilities" in 25 representative cities. These are facts taken from the records. *They show that 96% of all cast iron water mains ever laid in the 25 surveyed cities, since 1817, in sizes 6-inch and over, are still in service.*

The cast iron pressure pipe industry, in spite of raw material shortages, achieved in 1947 one of the biggest production years in footage in the history of the industry, and has recently installed additional footage capacity.

Mr. Q. Check says:
Collect and sell your scrap *NOW*

All statements and percentages regarding service records of cast iron mains set forth in this advertisement are based on published reports of a survey directed by a Joint Committee representing the American Water Works Association, New England Water Works Association, and the Institute of Water Supply Utilities. The findings, as they apply to cast iron pipe, have been reprinted by permission in our brochure "Survival and Retirement Experience with Cast Iron Water Mains." If you have not received a copy, write to Cast Iron Pipe Research Association, Thomas F. Wolfe, Engineer, 122 South Michigan Avenue, Chicago 3, Illinois.

CAST IRON

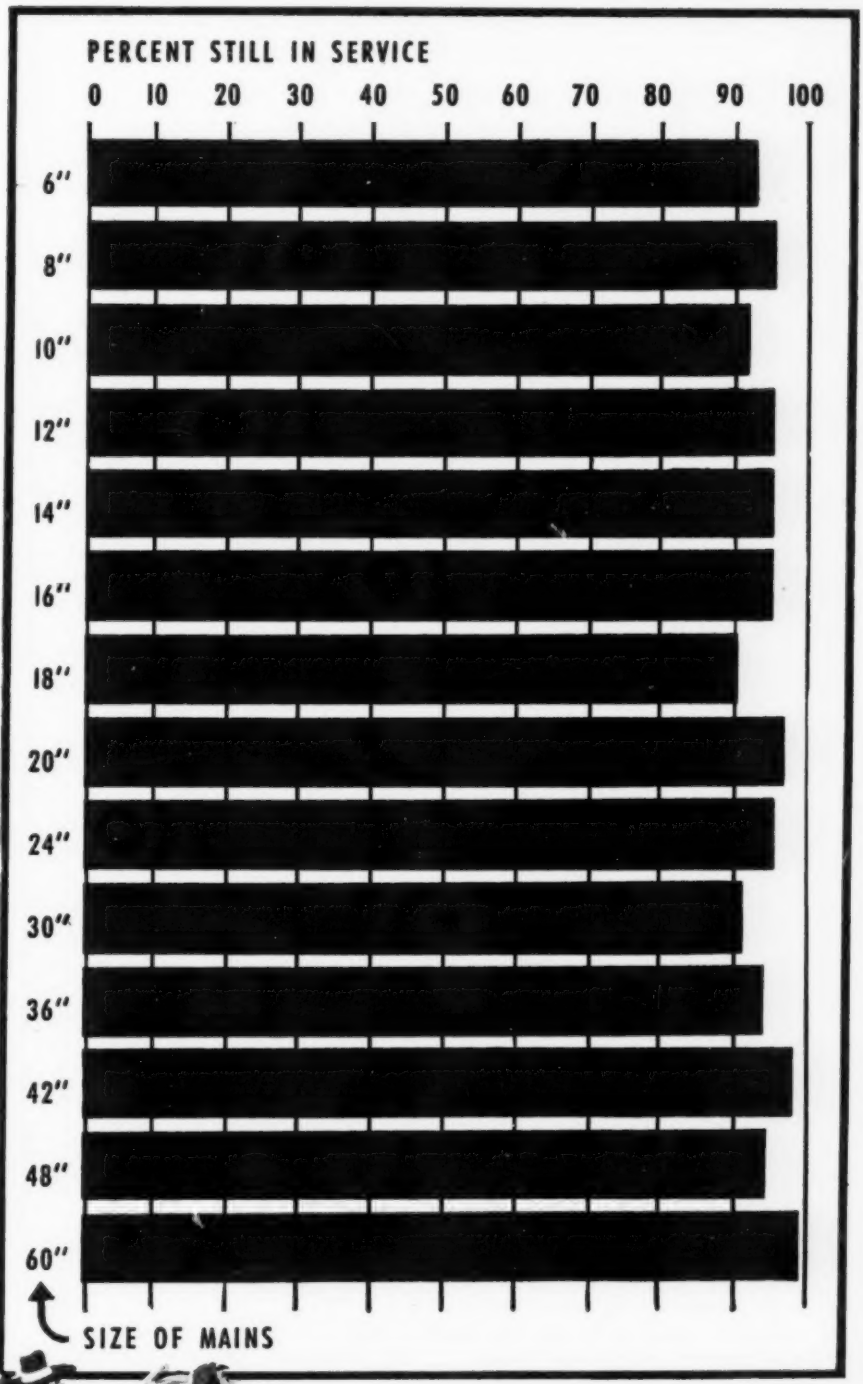
CAST IRON PIPE

HOW GOOD OUR PIPE IS!

96% OF ALL CAST IRON WATER MAINS* LAID IN THESE 25 CITIES OVER A PERIOD OF 128 YEARS ARE STILL IN SERVICE

Alexandria, Virginia
Babylon, New York
Clinton, Iowa
Clyde, New York
Denver, Colorado
Des Moines, Iowa
Detroit, Michigan
Huntington, West Virginia
Jamaica, New York
Merrick, New York
Norwich, New York
Ottawa, Ontario
Philadelphia, Pennsylvania
Portland, Maine
Rochester (Suburban), N. Y.
St. Mary's, Pennsylvania
St. Paul, Minnesota
Sag Harbor, New York
Scranton, Pennsylvania
Springfield, Massachusetts
Summit, New Jersey
Syracuse (Suburban), N. Y.
Utica, New York
West Palm Beach, Florida
Winnipeg, Manitoba

*Sizes from 6 to 60 inches.



SERVES FOR CENTURIES

Reduced Infiltration

DESIGNERS who have used Transite Sewer Pipe find that two factors—*tighter* joints and *fewer* joints—help to reduce ground water infiltration to an absolute minimum. Transite's sleeve-type joints make up tight and stay tight in service. Transite's long 13-foot lengths reduce the *number* of joints in the finished line.

These safeguards against infiltration cut down the volume which the line must carry. In addition, Transite has a high flow capacity (n=.010). This combination of reduced infiltration and better flow characteristics makes possible one of two immediate economies. Either smaller diameter pipe may be specified, or as an alternate saving, the use of flatter grades with correspondingly lower excavation costs may be considered.

In the long run, treatment costs are kept to a minimum . . . plant capacity conserved for the increased loads incident to future community growth.

Lower installation costs, too. Its long 13-foot lengths and fewer joints also make it easier to lay Transite Sewer Pipe to line and grade. Light in weight, it is readily handled. Many sizes, in fact, can be lowered into the trench without heavy mechanical handling equipment.

Transite Sewer Pipe is made of asbestos, cement and silica by a special process which develops maximum strength and uniformity, then steam cured to provide unusual resistance to corrosion. It is available in 4 strength classes in sizes to 36", permitting selection of the pipe best suited to trench conditions and often eliminating the need for concrete cradles. For further details, write Johns-Manville, Box 290, New York 16, N. Y.

*Reg. U. S. Pat. Off.

Johns-Manville

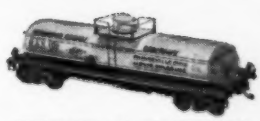
JOHNS-MANVILLE JM PRODUCTS

Transite Sewer Pipe

DIAMOND Liquid CHLORINE

will definitely help make this possible

Yes, pleasure boating, fishing, swimming and other recreational activities are a definite possibility—in nearby rivers and streams of pure, fresh water—as plans for the elimination of stream pollution progress. Diamond Liquid Chlorine is helping to make the over-all program possible, through the chlorination of domestic sewage and industrial wastes in disposal plants now being installed by communities and industries.



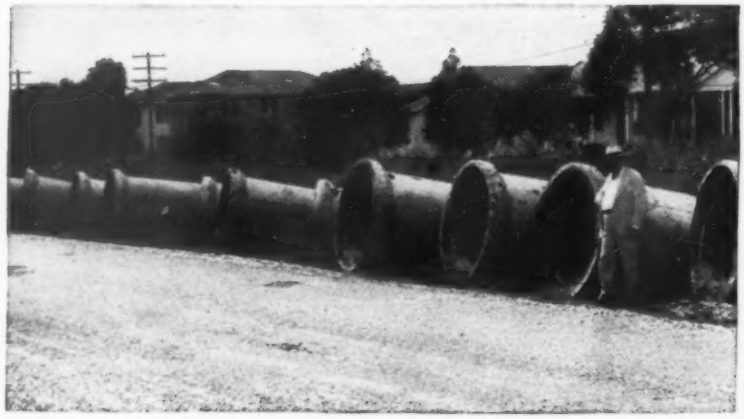
SUPPLIED IN TANK CARS

MULTI-UNIT CARS

OR CYLINDERS

CONCRETE PIPE laid in 1927....

Photos courtesy Sewerage and Water Board of New Orleans



Taken up and RE-USED IN 1947!

TWENTY YEARS ago New Orleans laid a 42″ and 48″ concrete pipe sewer in the bed of an old open drainage canal. In 1947 the city decided to build a wide covered drainage canal on the site. When the old concrete pipe was taken up, it was found to be in such excellent condition, that it will be re-used elsewhere in the city.

Thus is demonstrated again the durability, the structural strength and high wear resistance of concrete pipe. Concrete pipe provides maximum hydraulic capacity. Tight joints and uniformly dense concrete insure minimum infiltration and leakage. First cost is moderate. And as for life-long economy, consider the case of New Orleans!

AMERICAN CONCRETE PIPE ASSOCIATION

228 NORTH LASALLE STREET, CHICAGO 1, ILLINOIS

Painted for U. S. Pipe & Foundry Co. by Paul Laune

U.S. PIPE

THE quality of U. S. Cast Iron Pipe is controlled to a degree undreamed of by pipe founders a generation ago. Before the molten metal is poured into the mold of a Super-deLavaud centrifugal casting machine the important characteristics of the iron have been determined by our metallurgists. After the pipe has been cast our plant and research laboratories conduct numerous tests, in addition to acceptance tests, to assure the maintenance of quality in U. S. Pipe. This routine procedure at our several plants is enforced so that *we* may be confident that *your* confidence in our product continues to be justified. **United States Pipe and Foundry Co., General Offices: Burlington, New Jersey, Plants and Sales Offices Throughout U. S. A.**

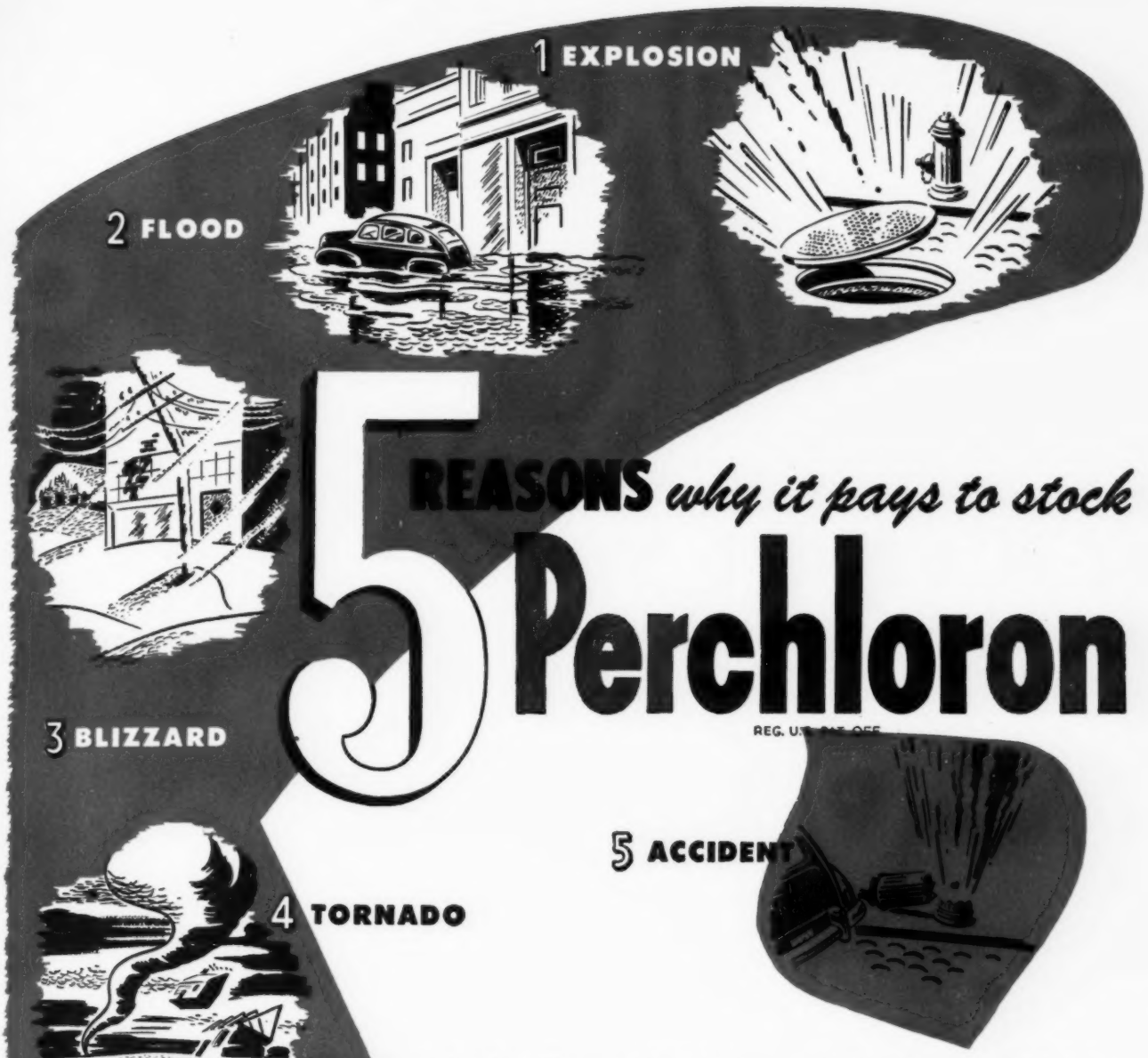
HELP FOR You IN THIS NEW BOOK!
REX CONVEYOR SLUDGE COLLECTORS
CHAIN BELT COMPANY of MILWAUKEE
SANITATION EQUIPMENT DIVISION
BULLETIN 47-9
CHECK THE "SPECS"
COMPARATIVE SPECIFICATIONS
Verti-Flo Clarifiers
for MULTIPLE tanks-primary or secondary with both longitudinal and cross-collectors
for SINGLE tanks-primary or secondary with longitudinal sludge collector only
REX conveyor
for MULTIPLE tanks-primary or secondary with both longitudinal and cross-collectors

YOU'LL FIND everything you want to know about sedimentation with Rex Conveyor Sludge Collectors in this new Booklet. Facts and figures that will help you plan your specifications are at your finger tips. Plan and sectional views of typical and special single and multiple primary and secondary tanks... set-ups for large and small plants... specifications... outstanding features... photographs of installations... data on auxiliary equipment... and many other important items are covered.

Bulletin No. 47-9 is a book that was written to help *you* select the right equipment for your jobs. Send for your free copy today. Address Chain Belt Company, 1610 West Bruce Street, Milwaukee 4, Wis.

CHAIN BELT REX COMPANY

SANITATION EQUIPMENT



5 REASONS *why it pays to stock* Perchloron

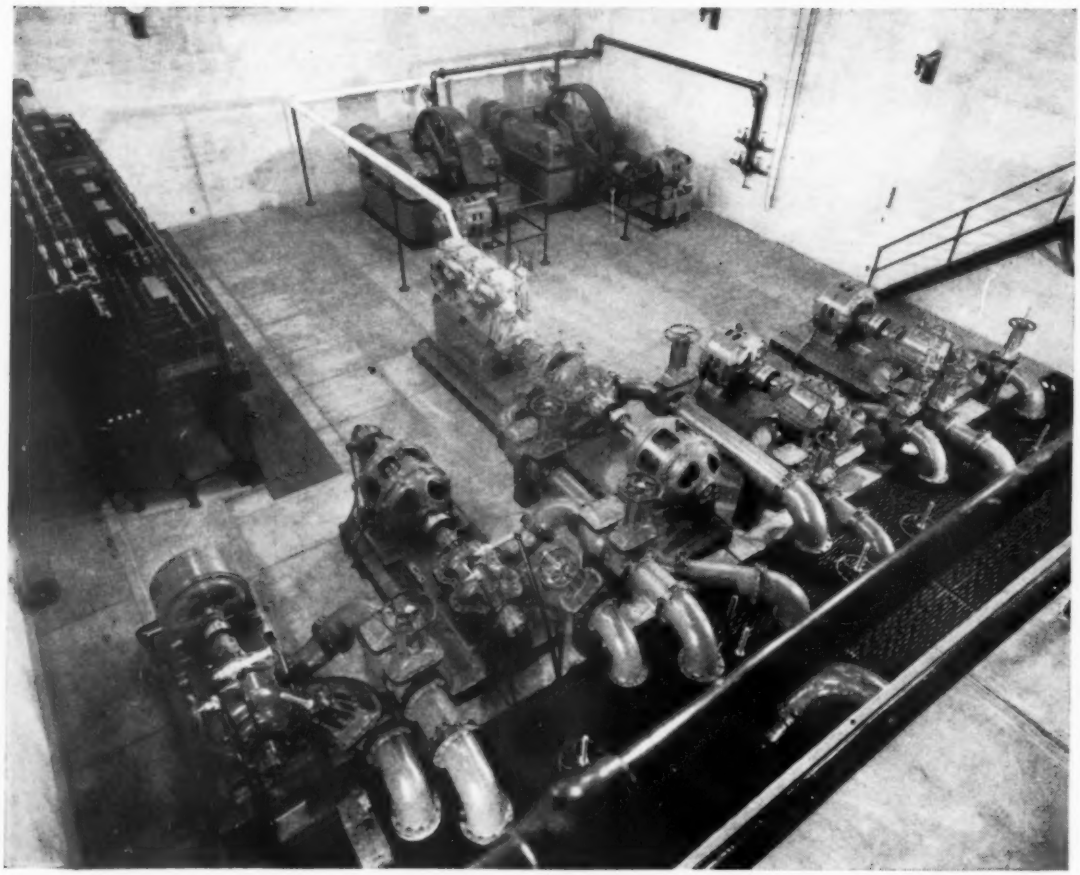
REG. U.S. PAT. OFF.

Yes, these sketches illustrate why so many alert engineers maintain a standby supply of Per*chlor*on.

They've found it pays to keep a stock of this handy calcium hypochlorite to help safeguard public water supplies in unforeseen emergencies. Per*chlor*on is stable and uniform. It contains over 70% available chlorine which means that 1 pound of Per*chlor*on is equal to 0.7 pounds of liquid chlorine.

Per*chlor*on is useful in other water treatment functions, too—sanitation of small water systems, new mains, wells, swimming pools. Its properties include the oxidation of ferrous iron in water, removal of hydrogen sulphide, control of most types of algae, and the destruction of chlorine-susceptible bacteria. Write for full descriptive material to Pennsylvania Salt Manufacturing Company, Philadelphia 7, Pa.

PENN SALT
CHEMICALS



Water Pumping Station, Fort Dodge, Iowa

IOWA GATE VALVES and CHECK VALVES

Specified for This Installation

Briefly, here is why you find "Iowa Valves" specified by Water Works Engineers, Superintendents and Operators of Water Treatment Plants, Pumping Stations and Distribution Systems, time after time after time: —

- **Recognized Design**
- **Quality of Materials**
- **Quality of Workmanship**
- **Prompt, Dependable Delivery**
- **35 Year history of satisfactory performance**

You are invited to fully acquaint yourself with current Iowa Valve developments and qualities. Write for completely illustrated, descriptive literature.

IOWA VALVE COMPANY

(Subsidiary of James B. Clow & Sons)

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To Serve The South...

GENERAL CHEMICAL ANNOUNCES

New Alum Facilities

at Jacksonville, Fla. and Macon, Ga.

To serve the South's rapidly growing paper, textile and other industries, General Chemical now has seven strategically-located producing Works. Latest to go into operation are Macon Works at Macon, Ga. and Jacksonville Works at Jacksonville, Fla., both producing Aluminum Sulfate. These, together with other extensive facilities at Savannah and Atlanta, Ga., New Orleans, Baton Rouge, and Monroe, La., provide a convenient source of supply to meet your requirements for "Alum," and the many other basic chemicals General Chemical produces for American Industry.

Operating in close conjunction with these works are General's regional offices and distributing stations at Charlotte, N. C., Atlanta, Ga. and Birmingham, Ala., a combination that assures you of supplies and service—swiftly and steadily at all times.

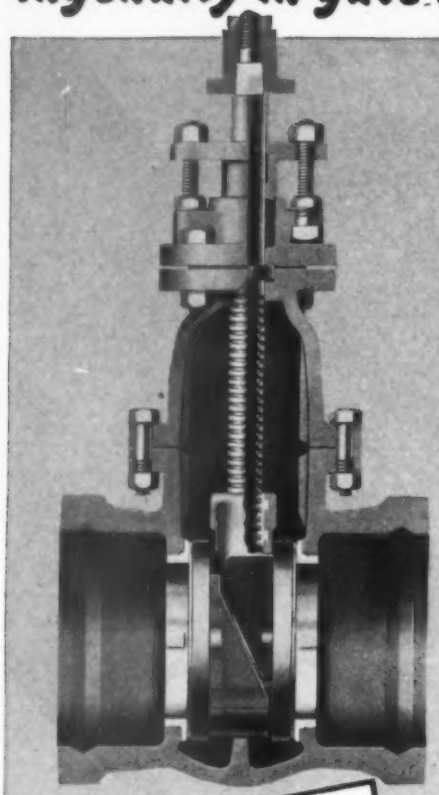
GENERAL CHEMICAL DIVISION
ALLIED CHEMICAL & DYE CORPORATION
40 Rector Street, New York 6, N. Y.
Regional Offices: Atlanta, Ga. • Birmingham, Ala. • Charlotte, N. C.

BASIC CHEMICALS

GC

FOR AMERICAN INDUSTRY

*Ingenuity in gate valve design fulfills seven objectives**



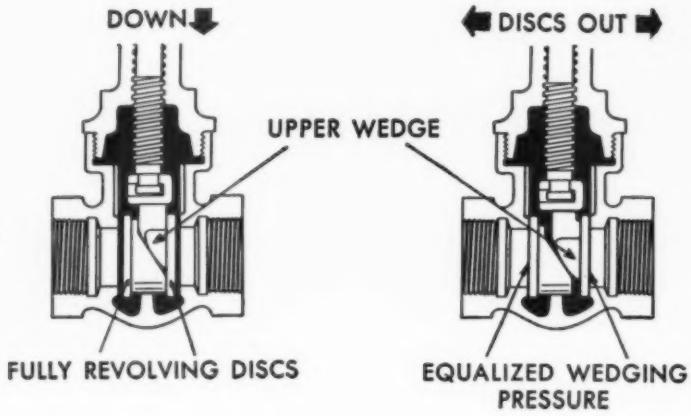
*This advertisement is one of a series showing how the unique Darling Fully Revolving Double Disc Parallel Seat Gate Valves answer the following critical operating problems:

- Positive, easy closing—regardless.
- Automatic adjustment for valve body distortion.
- Elimination of disc-to-seat friction and galling.
- Uniform wear distribution.
- Extreme simplicity.
- Greater service life.
- Simplified maintenance.

Darling Parallel Seat Gate Valves are available in a wide range of sizes and for all types of normal and unusual service, and for pressures up to 3000 pounds. Likewise, Darling Taper Seat, Solid or Slotted Wedge Gate Valves are available for most services.

Corrosion resistant valves. *In addition to conventional iron, bronze and steel types, Darling specializes in valves for any corrosive fluid: iron body valves with special alloy trim; iron body, rubber lined, with special alloy trim; all bronze; and all special alloy. Darling's 60 years of experience in meeting unusual requirements is always at your service.*

POSITIVE, EASY CLOSING ...REGARDLESS!



Regardless of years of hard, continual service, involving anything from extreme disc and seat wear to actual body distortion, this valve continues to give positive, leak-proof closure . . . and without strong-arm methods . . . an advantage found only in Darling Fully Revolving Double Disc Parallel Seat Gate Valves.

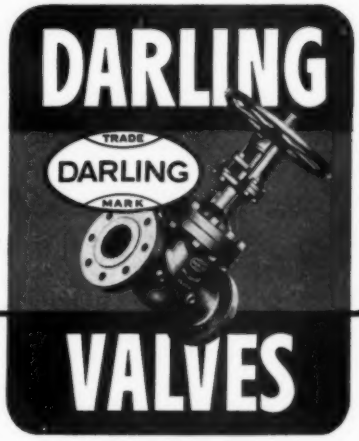
Note how the face of the upper wedge is radiused, while the adjacent face of the lower wedge is straight. Thus wedging pressure on the discs is automatically equalized, always uniform. If, because of body distortion, the valve seats become unparalleled, this unique compensating wedge principle still provides perfect seating and tight closure without forcing.

The discs themselves are free to revolve fully and seat in a different position at every closing. Uneven wear, another common cause of leaks, is definitely avoided.

Combine these unique features—automatic seating adjustment and uniform wear—and it's easy to see why these Darling Valves can always be counted on for positive, easy closing—regardless!

DARLING VALVE & MANUFACTURING CO.
Williamsport 11, Pa.

Outline your needs and ask for specific data on the proper Darling Valves... or send for the complete 300-page Darling Catalog No. 17M. It describes Darling Valves of *all* types and is full of valuable information.



THE VALVE MARK OF QUALITY:::WATCH FOR IT

we embark on our

with pride in our past accomplishments, and confidence in our ability to render an ever increasing service in the field of water purification and sanitation

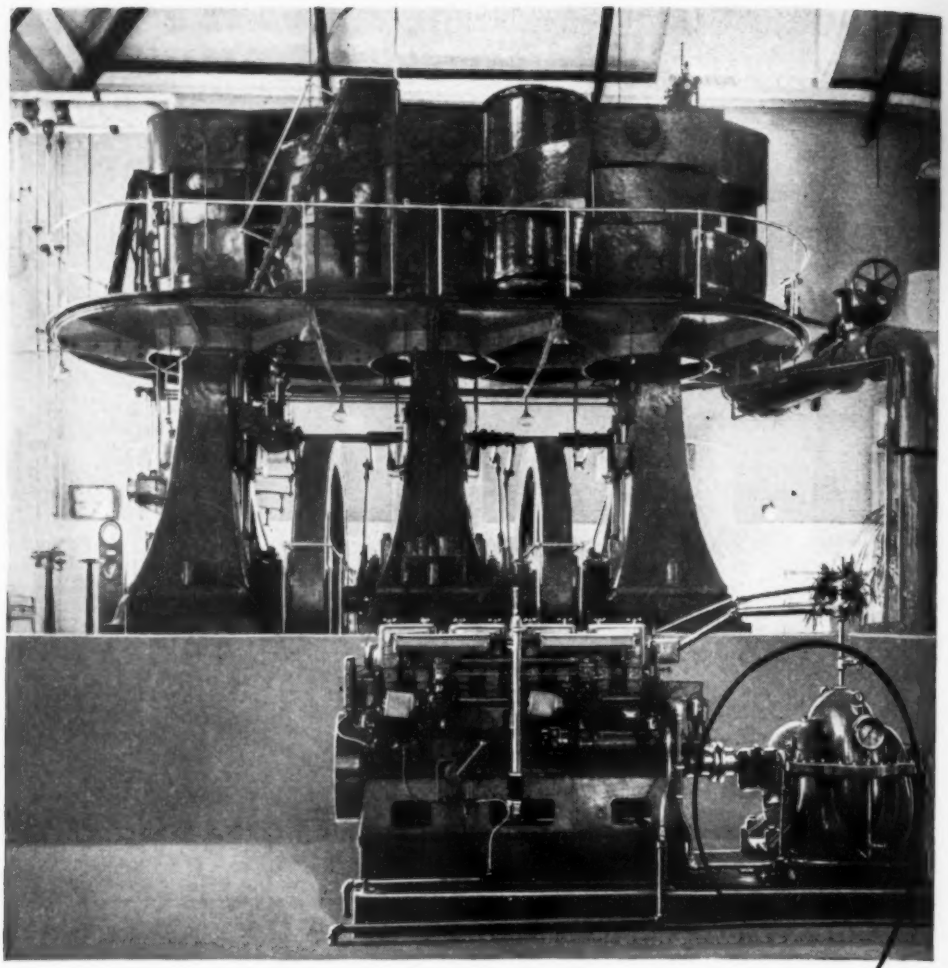
1897

MECHANICAL EQUIPMENT
BY
ROBERTS FILTER MFG. CO.
DARBY, PENNA.

1947

ROBERTS FILTER MANUFACTURING CO.

607 COLUMBIA AVE., DARBY, PA.



New De Laval Centrifugal Pump

REPLACES OBSOLETE GIANT

as Atlantic City modernizes

Replacement of the old triple-expansion pumping engine, shown in the background with the new De Laval centrifugal pump in the foreground, is part of the modernization program for Atlantic City. This station now employs a total of seven De Laval centrifugal pumps having a combined capacity of more than 73 million gallons daily. Four of these are motor-driven pumps employed for practically continuous operation while three are gasoline engine-driven pumps employed for stand-by service. All seven units are equipped with self-priming systems so that they can pick up the load upon a minute's notice.

DE LAVAL

DE LAVAL STEAM TURBINE CO., TRENTON 2, N. J.

Atlanta • Philadelphia • Los Angeles • Chicago • Pittsburgh • San Francisco
St. Paul • Cleveland • Tulsa • Boston • Charlotte • Detroit • Seattle • Toronto
New York • Kansas City • Vancouver • Rochester • New Orleans • Edmonton
Denver • Salt Lake City • Winnipeg • Helena • Houston • Washington, D. C.

WW-6

TURBINES • HELICAL GEARS • WORM GEAR SPEED REDUCERS • CENTRIFUGAL PUMPS • CENTRIFUGAL BLOWERS AND COMPRESSORS • IMO OIL PUMPS

AT THE A.W.W.A. CONVENTION, VISIT DE LAVAL BOOTHS NUMBER 77 AND 78.

Next stop . . .

ATLANTIC CITY!

WE OF THE R. D. WOOD Co. are particularly glad that Atlantic City was chosen for this year's AWWA Convention. We always have a good time there. Who doesn't? And as it is so close to our home town, perhaps some of you will come up and see us when the big jamboree is over. We hope you do.

Look for us at Booths 24 & 25. We will have a model of today's great Mathews Hydrant on display, together with complete information about our other products. We assure you a warm welcome, for we are always glad to see those who are engaged in this vital industry.

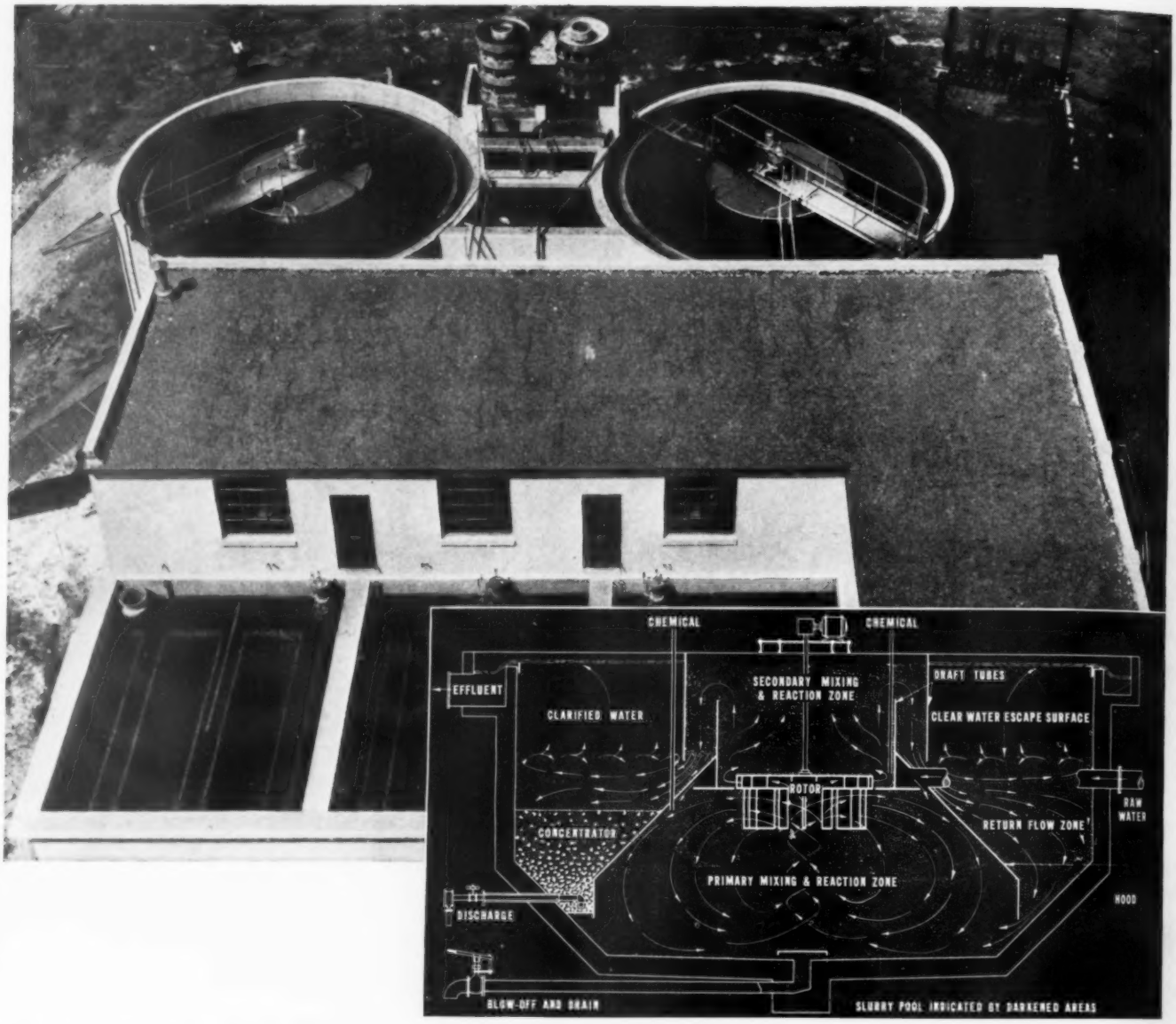
See you, then, at Booths 24 & 25, Convention Hall, in hospitable and fun-loving Atlantic City. Remember the dates: May 3-7.

MATHEWS HYDRANTS

Made by R. D. WOOD COMPANY

Public Ledger Bldg., Independence Square, Philadelphia 5, Pa.

Makers also of "Sand-Spun" Pipe (centrifugally cast in sand molds) and R. D. Wood Gate Valves



ACCELATOR HAS THE HIGHEST RATE OF TREATED WATER DELIVERY

The ACCELATOR* delivers treated water faster than any other type of water treating equipment because of two exclusive advantages:

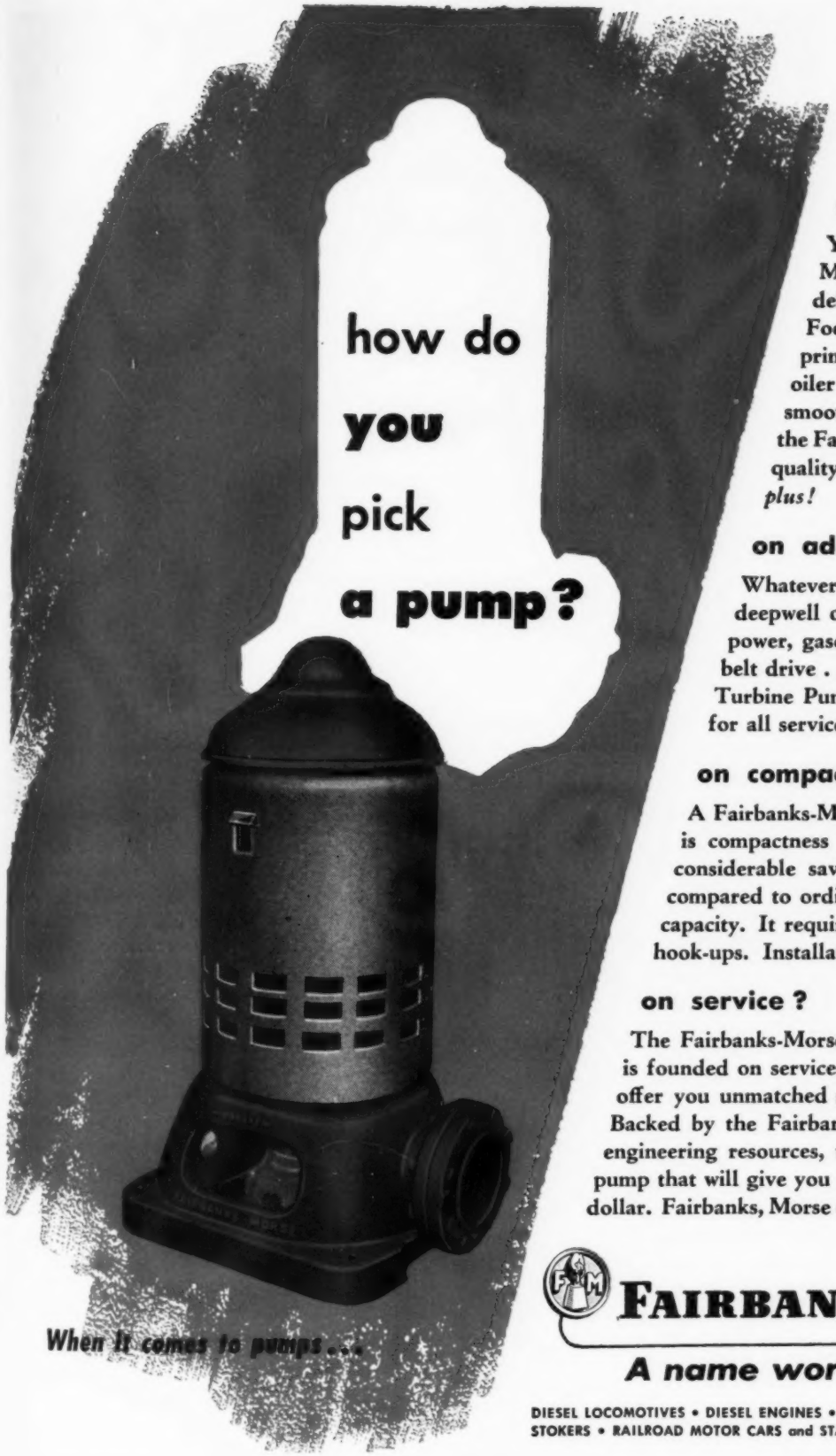
INSTANTANEOUS CHEMICAL REACTION takes place on the surface of old particles of an already well coagulated slurry with which the raw water and treating chemicals are intermingled. The slurry concentration in the mixing zone of the ACCELATOR is greater than in any other type of water treating equipment, due to the positive return of solids from the separation zone.

"DYNAMIC" SEPARATION occurs in the ACCELATOR when clear, treated water separates and rises almost at right angles from the surface of the lateral flow of slurry. There is no settling of solids . . . no percolating or filtering up through a suspended sludge blanket.

Send for ACCELATOR Bulletin 1824. INFILCO Inc., 325 West 25th Place, Chicago 16, Illinois.

*Trade-Mark Reg. U. S. Pat. Off.

IF YOU WANT TO KNOW — ASK INFILCO



on dependability?

Year after year, Fairbanks-Morse Vertical Turbine Pumps deliver outstanding performance. Fool-proof operation requiring no priming . . . automatic solenoid oiler . . . precision construction for smoother flow . . . and backed by the Fairbanks-Morse reputation for quality, they assure dependability *plus!*

on adaptability?

Whatever the job requirements . . . deepwell or close-coupled, electric power, gasoline or Diesel engine, or belt drive . . . Fairbanks-Morse Vertical Turbine Pumps are readily adaptable for all service conditions.

on compactness?

A Fairbanks-Morse Vertical Turbine Pump is compactness itself . . . permits considerable savings in floor space as compared to ordinary pumps of comparable capacity. It requires no complicated piping hook-ups. Installation is easy and economical.

on service?

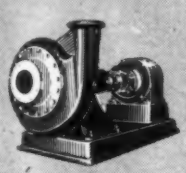
The Fairbanks-Morse Pump Dealer organization is founded on service. They *know* pumps . . . can offer you unmatched service and assistance. Backed by the Fairbanks-Morse production and engineering resources, they can help you pick the pump that will give you most for your pumping dollar. Fairbanks, Morse & Co., Chicago 5, Illinois.

FAIRBANKS-MORSE

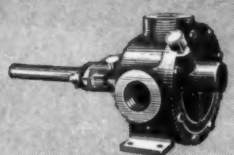
A name worth remembering

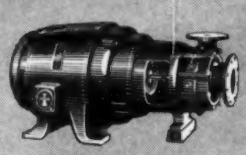
DIESEL LOCOMOTIVES • DIESEL ENGINES • PUMPS • SCALES • MOTORS • GENERATORS
STOKERS • RAILROAD MOTOR CARS and STANDPIPES • FARM EQUIPMENT • MAGNETOS

A FEW OF THE COMPLETE LINE OF FAIRBANKS-MORSE PUMPS

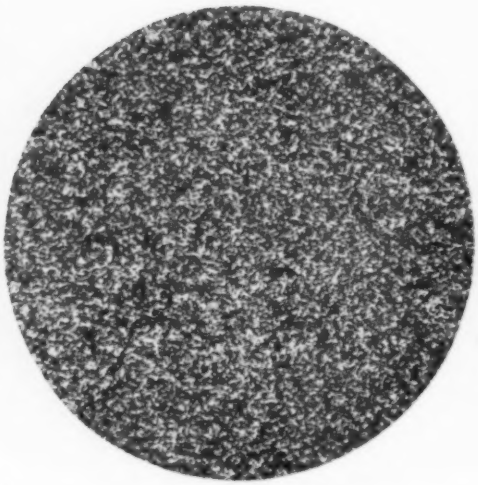

Trash Pump

Side-Suction


Rotary


Builtogether

Making Sewage Aeration More Effective with ALOXITE Diffusers



Close-up of Diffuser surface reveals homogeneous hard structure.

Photomicrograph (X55) reveals uniform blocky grains and evenly distributed pores.

Unique Structure Speeds release of Small Bubbles

ALOXITE electrically fused aluminum oxide Diffuser Media are made from carefully selected sizes and shapes of grain bonded together with a special ceramic binder, molded under pressure and vitrified at high temperatures.

Microscopic examination shows uniform, blocky, angular edged grain and evenly distributed pores of ample cross section. Smaller air bubbles are quickly released from such a surface because their surface tension is less. High permeability, controlled in manufacture to assure uniformity, minimizes clogging ... offers low resistance to air passage. Treatment is thus effective.

Structure is stable and durable. It resists acids and corrosion. It withstands temperature changes and physical or mechanical stresses. ALOXITE plates and tubes are easily cleaned ... simple to install quickly.

Our engineering staff is technically qualified to advise on specification, installation and operation of ALOXITE diffusers. They have plenty of practical experience in the sewage field. Let them help you increase the effectiveness of your water or sewage treatment. They will be glad to cooperate. Write Department M-48, The Carborundum Company, Refractories Division, Perth Amboy, New Jersey.

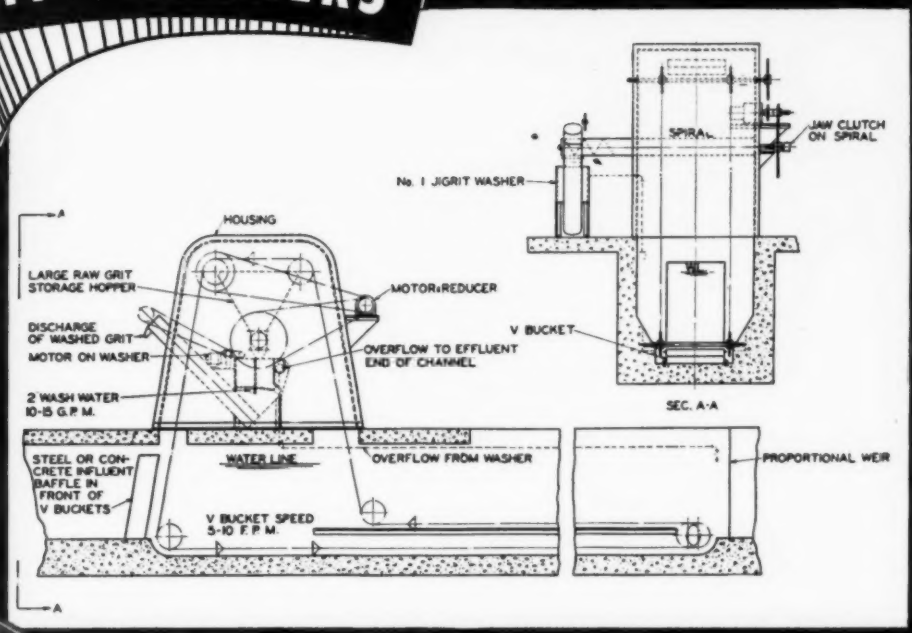
DIFFUSER MEDIA

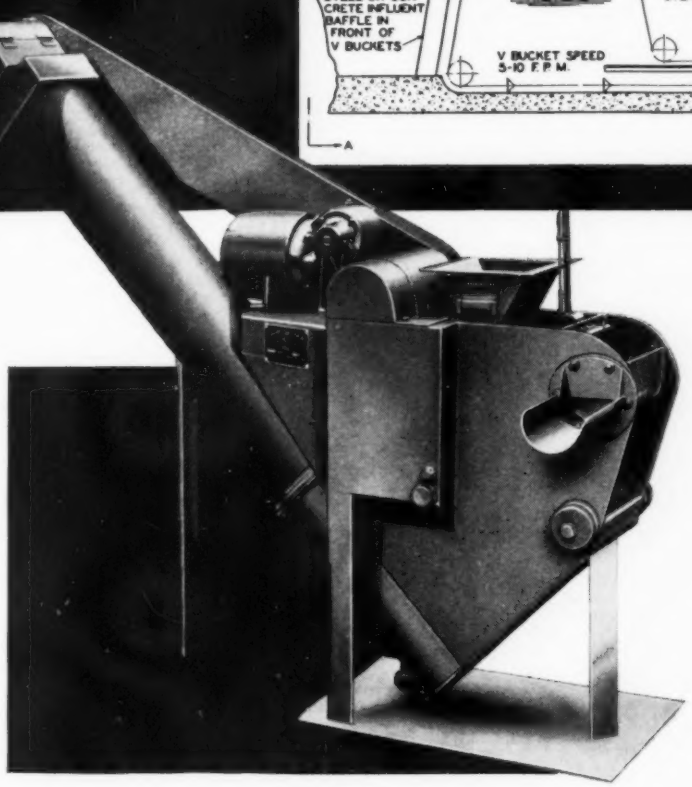
UNIFORM STRUCTURE
CORROSION RESISTANT
RUGGED AND DURABLE
ECONOMICAL
EASY TO CLEAN
LOW PRESSURE LOSS

BY **CARBORUNDUM**
TRADE MARK

"Carborundum" and "Aloxite" are registered trademarks which indicate manufacture by The Carborundum Company

85% REMOVAL OF ORGANICS with "JIGRIT" WASHERS





Left — Jeffrey "JIGRIT" No. 1 — a basket type jig for classifying and removing putrescible solids from sewage grit in plants of moderate size. Capacity — 20 cubic feet per hour. 90% recovery of inorganic grit. Produces a washed grit that can be used for fill or dumped anywhere without nuisance. Also No. 4 "JIGRIT" washer for washing grit at rate of 120 cubic feet per hour, and No. 9 for 280 cubic feet per hour . . in single units fully automatic in operation.

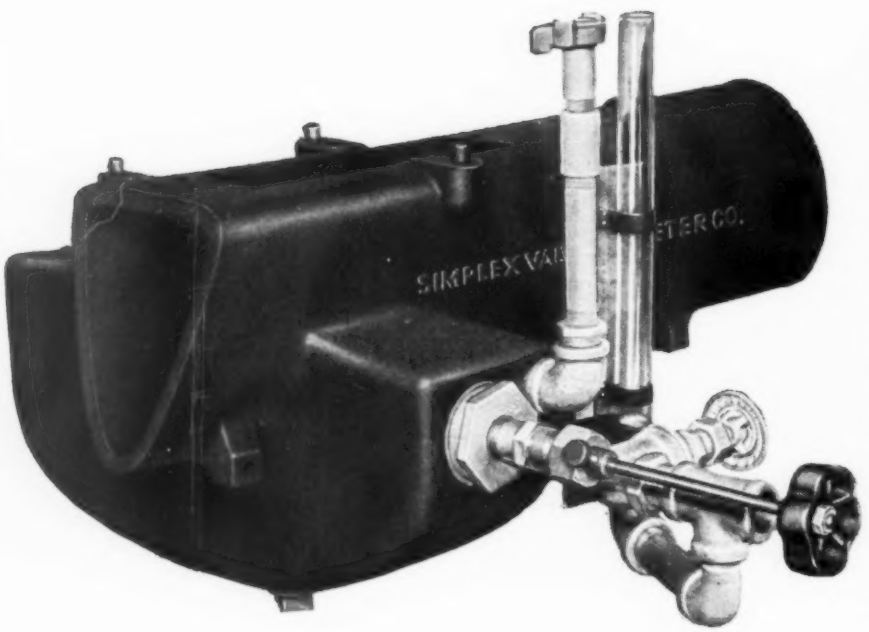
Bar Screens • Sludge Collectors • Sludge Elevators • Grit Collectors • Screenings Grinders
Garbage Grinders • Conveyors, Chains and Bearings • also equipment for biofiltration plants

JEFFREY

ESTABLISHED 1877

JM

MEASURES EXTREME LOW FLOW OF WATER, SEWAGE AND INDUSTRIAL LIQUIDS



Simplex Type "S" Parabolic Flume is designed to measure accurately flows of all kinds of liquids through partially-filled pipes, sewers or channels where free-flowing conditions exist. The Simplex "S" Flume permits measurement of fluid under low head conditions and with small head loss . . . needs only slight free fall of the liquid at flume outlet. Write for bulletin and full information to the Simplex Valve & Meter Company, 6743 Upland Street, Philadelphia 42, Pa.

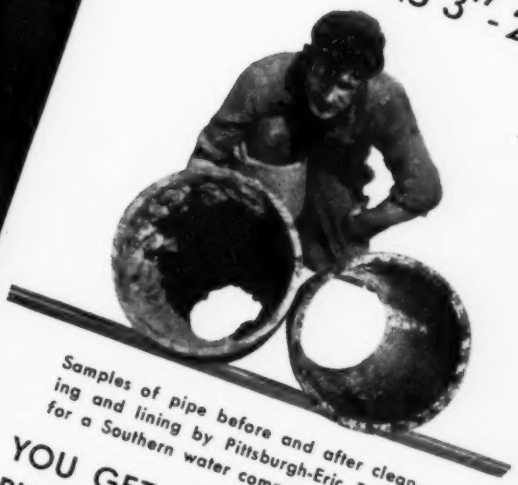
SIMPLEX

VALVE AND METER COMPANY

PITTSBURGH-ERIC
PIPE RECONDITIONING

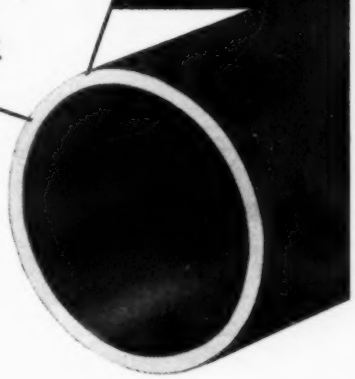
- PROLONGS LIFE OF SMALL WATER MAINS
- RESTORES FULL CAPACITY
- MAINTAINS HIGH VOLUME
- ELIMINATES RED WATER
- ECONOMICALLY APPLIED
- PRESERVES EQUITY OF WATER MAIN INVESTMENT

FOR WATER MAINS 3"-24" I.D.



Samples of pipe before and after cleaning and lining by Pittsburgh-Eric Process for a Southern water company.

YOU GET FULL VOLUME PLUS PIPE PROTECTION . . . when mains are cleaned and lined by PITTSBURGH-ERIC PROCESS.



Expert Pipe Cleaning Combined With Eric Process of Lining Pipe Electrolytically Makes Small Water Main Rehabilitation Programs Available At Low Cost.

- As much as 1,000 feet of pipe can be cleaned and lined at one time.
- Under normal conditions the main can be restored to service the same day.
- Removal of corporation cocks is not required.
- More than 2,500,000 feet of pipe have been coated by the Eric Process in ten years.

Get the full details about a Pittsburgh-Eric Pipe Reconditioning Program for your community. Ask our nearest office TODAY!

LECTUMEN
PIPE LINING

LECTUMEN is an emulsified solution of bitumen especially prepared for use with Pittsburgh-Eric Process. Imparts a smooth, long-lasting, corrosion resistant coating to steel and C. I. pipe interiors.

Because LECTUMEN is applied electrolytically to the pipe interior, it is necessary that the main be cleaned DOWN TO THE METAL before the coating operation is accomplished. Pittsburgh-Eric specifications set a higher standard of CLEAN-NESS than is ordinarily required. Fullest capacity is thus restored.

PITTSBURGH PIPE CLEANER COMPANY
133 Dahlem St., Pittsburgh 6, Pa.

PHILADELPHIA · BALTIMORE · WASHINGTON · NEW YORK · BUFFALO · CHICAGO · CINCINNATI · ST. LOUIS · DETROIT · BOSTON

Worthington-Gamon WATCH DOG WATER METERS

The meter used by thousands of municipalities in the U. S. and abroad.

SURE TO MEET YOUR SPECIFICATIONS FOR ACCURACY, LOW MAINTENANCE, LONG LIFE

Before you invest in water meters, get acquainted with the design and performance advantages which make Worthington-Gamon Watch Dog Water Meters first choice of so many municipalities and private water companies in the United States.

WATCH DOG WATER METERS

"Watch Dog" models . . . made in standard capacities from 20 gpm up; frost-proof and split case in household sizes. Disc type, Turbine type or Compound type. Write for Bulletin.

WORTHINGTON-GAMON METER COMPANY

296 South Street, Newark 5, New Jersey

Subsidiary of

WORTHINGTON

PUMP AND MACHINERY CORPORATION

Super-Market for PIPING

● **A GRINNELL WAREHOUSE** can be your *one supplier* of piping . . . a dependable source for *all* your requirements.

In addition to the broad line of Grinnell-manufactured piping products, each Grinnell Warehouse carries a complete stock of pipe, valves, and piping specialties . . . and can help you engineer your piping system. You save time and money when you "shop" at Grinnell.

Take advantage of this *complete* piping service . . . call on Grinnell "whenever piping is involved".

GRINNELL OFFERS A COMPLETE PIPING SERVICE

PIPING SUPPLIES

- Pipe, Valves and Fittings
- Engineered Pipe Hangers
- Grinnell-Saunders Diaphragm Valves
- Thermolier Unit Heaters
- Prefabricated Piping
- Plumbing and Heating Specialties
- Oil, Water Works and Industrial Piping Supplies

OTHER GRINNELL PRODUCTS

Automatic Sprinklers and Special Hazard Fire Protection Systems

AMCO Humidification and Cooling Systems

GRINNELL COMPANY, INC.
Executive Offices
PROVIDENCE 1, RHODE ISLAND

GRINNELL

WHENEVER **PIPING** IS INVOLVED

You are cordially invited to visit the
Calgon booth, No. 60, at the A. W. W. A.
Convention, Auditorium, Convention Hall,
Atlantic City, May 3-7, 1948
See for yourself how Calgon*
prevents scale . . . controls corrosion . . .
stabilizes lime-softened water . . .
eliminates "red water"

o

*T.M. Reg. U. S. Pat. Off.

HAGAN
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BUROMIN
CALGON

calgon, inc.
A SUBSIDIARY OF
HAGAN CORPORATION
HAGAN BUILDING
PITTSBURGH 30, PA.

Glauber

GROUND-KEY WORK Has Maintained a Reputation of Quality Since 1890

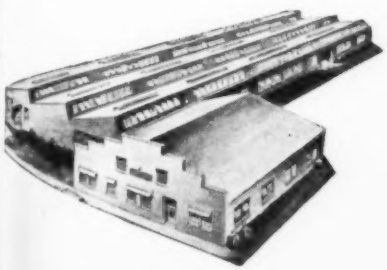
Engineers who have specified Glauber Brass products for installation in their community water systems during the last fifty years have every reason to be gratified by their choice. Years of research, modern manufacture and individual testing of each fitting under 200 pounds constant pressure guarantee that when a Glauber ground-key unit is installed it can be expected to operate for the lifetime of the installation of which it becomes a part. Behind every Glauber product are 58 years of manufacturing experience and "know how".

The Glauber line includes many features upon which water works engineers have learned to depend for years of trouble-free service. Precision ground-key valve parts are lubricated by special non-deteriorating grease containing only vegetable oils. The Inverted Key Curb Stop is designed so that the bypass admits the full force of the water beneath the plug, thereby creating a tighter seal under high pressures,

Use This New Catalog as your Guide in Specifying Ground-Key Work

and in turn eliminating all possibilities of key failures. Each piece of Glauber ground-key work has a mechanically perfect taper and water-tight bearing.

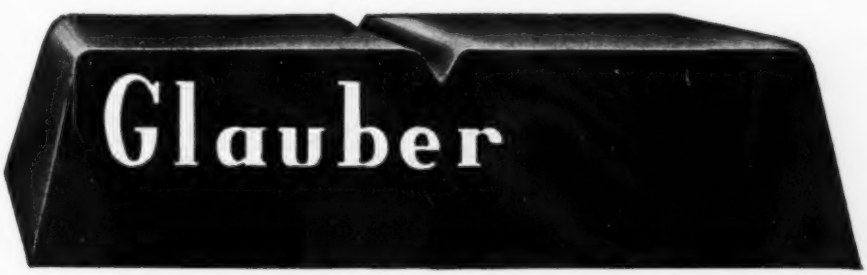
The standard of quality built into each product of the Glauber Brass Manufacturing Company assures the Water Works Engineers and City Superintendents who specify the Glauber line that their community will be well served.

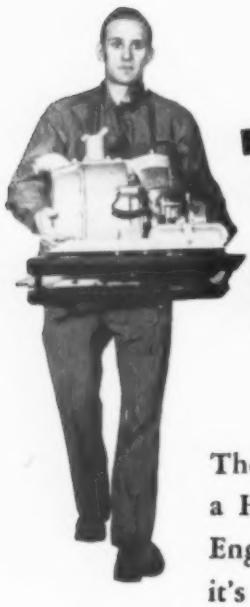


Water works superintendents and engineers, a copy of this new catalog is your for the asking. It describes in detail the complete line of Glauber Ground-Key and Water Works Fittings which have earned a reputation for dependable operation and service over the past 58 years. All types, including hand wiped lead goosenecks are available in quantities to meet normal delivery requirements. Start now to specify Glauber ground-key work for both new and replacement installations. All units are individually tested to assure operating perfection. You and your associates will also like the attention and prompt cooperative service given your proposals for prices and the delivery of ultimate orders.

THE Glauber BRASS MFG. CO., KINSMAN, OHIO

Division of H. B. SALTER MFG. CO., Marysville, Ohio





The One Carryable Pump for all jobs

There's one thing about a Homelite Gasoline-Engine-Driven Pump... it's small enough to get to a job and big enough to *do* the job. But that's not all... not by far. A Homelite... *and only a Homelite* ...has 9 Big Features that make it the best pump for all jobs... your jobs.

The Best Pump for All Jobs is the Pump with All These Features		
1	CARRYABILITY	√
2	FAST SELF-PRIMING	√
3	AUTOMATIC SEEPAGE CONTROL	√
4	HIGH 28-FOOT SUCTION LIFT	√
5	NON-CLOGGING DESIGN	√
6	MAINTAINED RATED CAPACITY	√
7	DISCHARGE PRESSURE	√
8	DEPENDABILITY	√
9	SIMPLE DESIGN AND OPERATION	√

Handles Mud and Sand Easily

Non-clogging, self-scouring, with no pockets to accumulate solids, a Homelite *Carryable* Pump handles the thickest and muddiest water without stoppage or shutdowns for cleaning.

See for yourself

Write today for our new illustrated bulletin No. L-503 that tells the *complete* story and shows how you can get faster, better pumping with Homelite *Carryable* Pumps.

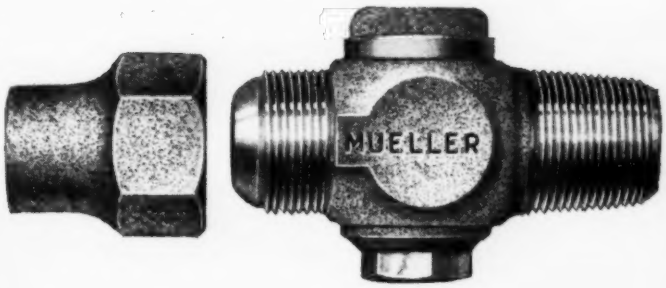
HOMELITE
CORPORATION
704 RIVERDALE AVENUE, PORT CHESTER, NEW YORK

MANUFACTURERS OF HOMELITE *CARRYABLE* PUMPS · GENERATORS · BLOWERS · CHAIN SAWS

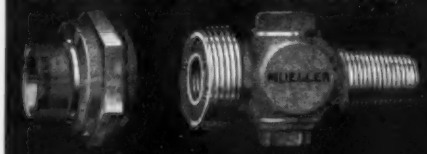
What Type Connections Do You Need?

MUELLER THREAD INLET
WIPED JOINT OUTLET

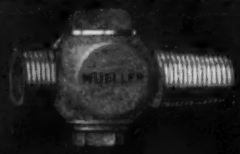
MUELLER THREAD INLET
COPPER PIPE OUTLET



MUELLER THREAD INLET
LEAD FLANGE OUTLET



MUELLER THREAD INLET
IRON PIPE OUTLET



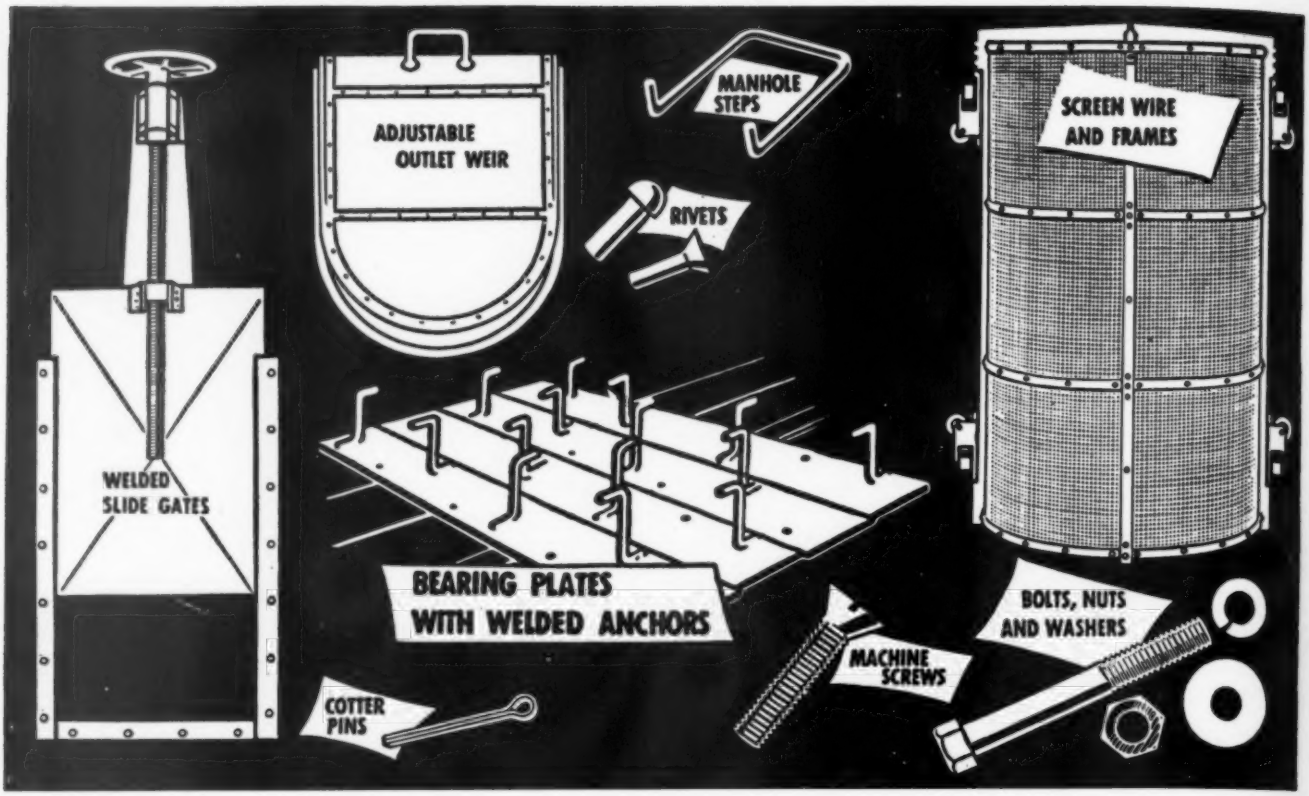
ANY OF THESE STOPS CAN BE INSERTED WITH A MUELLER TAPPING MACHINE

Regardless of the type connections you require on your Corporation Stops, you'll find them available in the Mueller Line with a correctness of design, proper balance of metal formulae, accurate machining and testing---all factors that have given Mueller Corporation Stops an enviable reputation for the highest degree of reliable service.

Mueller Corporation Stops are obtainable in a variety of styles, patterns and end connections to meet the requirements of standardized waterworks practice. Outlets may be for Iron, Copper or Lead connections with the Inlets for Iron, Steel, Copper or Transite Pipe. Consult your Mueller Water Works Catalog "H" or write us concerning your requirements.

MUELLER CO.

MAIN OFFICE AND FACTORY.....DECATUR, ILLINOIS
OTHER FACTORIES: *Los Angeles, Cal.; Chattanooga, Tenn.; Sarnia, Ont. Canada*



If your sewage system handles industrial wastes...

CONSIDER *Everdur*

SEWAGE SYSTEMS that handle industrial as well as domestic wastes need extra protection from corrosion. Everdur* Copper-Silicon Alloys are used extensively for this service.

For Everdur Alloys have excellent resistance to corrosion at all stages of sewage treatment operation. In addition, depending on the type or composition, Everdur Alloys may be worked hot or cold, have good machining properties and are readily formed, forged and welded. Everdur thus is readily adaptable to the fabrication of lightweight built-up equipment which offers exceptional durability and ease and economy of operation.

Consider specifying Everdur for such units as gates, screens, weirs, troughs, hoppers, orifices, conduit . . . even bolts, nuts and cotter pins wherever such small but important items are subject to corrosive influences. It can pay handsome dividends in trouble-free, long-term operations.

For more detailed information, write for Publications E-5 and E-11. 48592

*Reg. U. S. Pat. Off.

ANACONDA
from mine to consumer

Everdur

COPPER-SILICON ALLOYS

THE AMERICAN BRASS COMPANY

General Offices: Waterbury 88, Connecticut

Subsidiary of Anaconda Copper Mining Company

In Canada: ANACONDA AMERICAN BRASS LTD.
New Toronto, Ont.

ZECO GREENSAND ZEOLITE

ZECO and HI-ZECO Greensand Zeolite for water softening, filtration and iron removal. ZECO Manganese Zeolite for iron and manganese removal. COREXITE mineral for corrosion control and water stabilization.

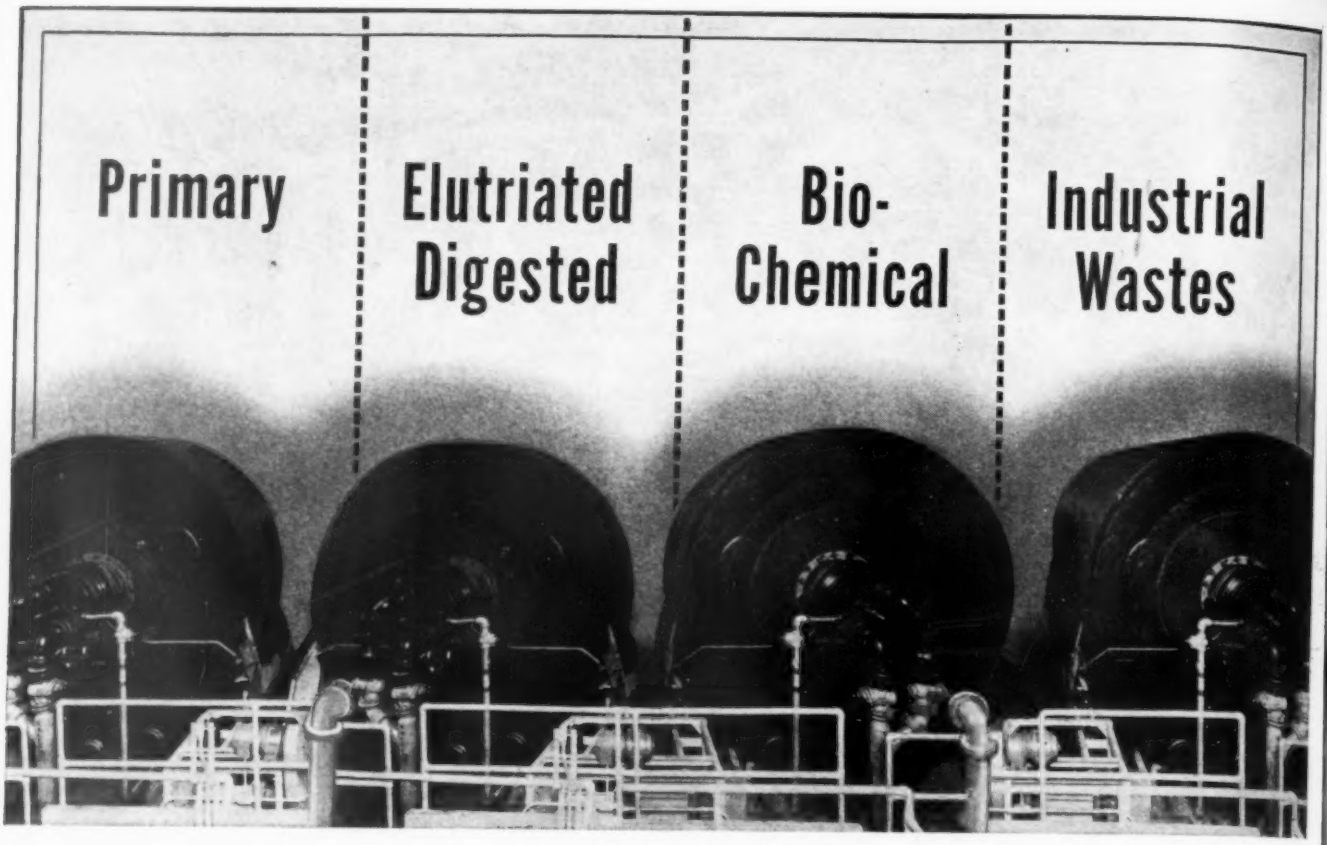
★

ZEOLITE CHEMICAL CO.

Sales Office
140 Cedar St., New York, N. Y.

Factory
Medford, N. J.

Pioneer Producer of GREENSAND ZEOLITE MINERAL



OVER 1000 TONS OF DRY SOLIDS ARE DEWATERED DAILY

The ability of Conkey Sludge Filters to effectively dewater various types of sludges and wastes—in large or small communities—has been proved for many years.

Municipal and consulting engineers should let General American engineers work with them when planning extensions to present dewatering facilities or constructing new plants. They will benefit from the experience of engineers who have a correct understanding of the vital factors which assure an efficient and economical installation, such as, feed consistency, conditioning of chemicals, conditioning time, application and mixing, as well as the all important knowledge of filter construction.

Other
GENERAL AMERICAN
EQUIPMENT

SLUDGE DRYERS
SLUDGE GAS HOLDERS
STEEL STACKS
TURBO-MIXERS
STORAGE TANKS
THICKENERS
SCREENING DEWATERERS

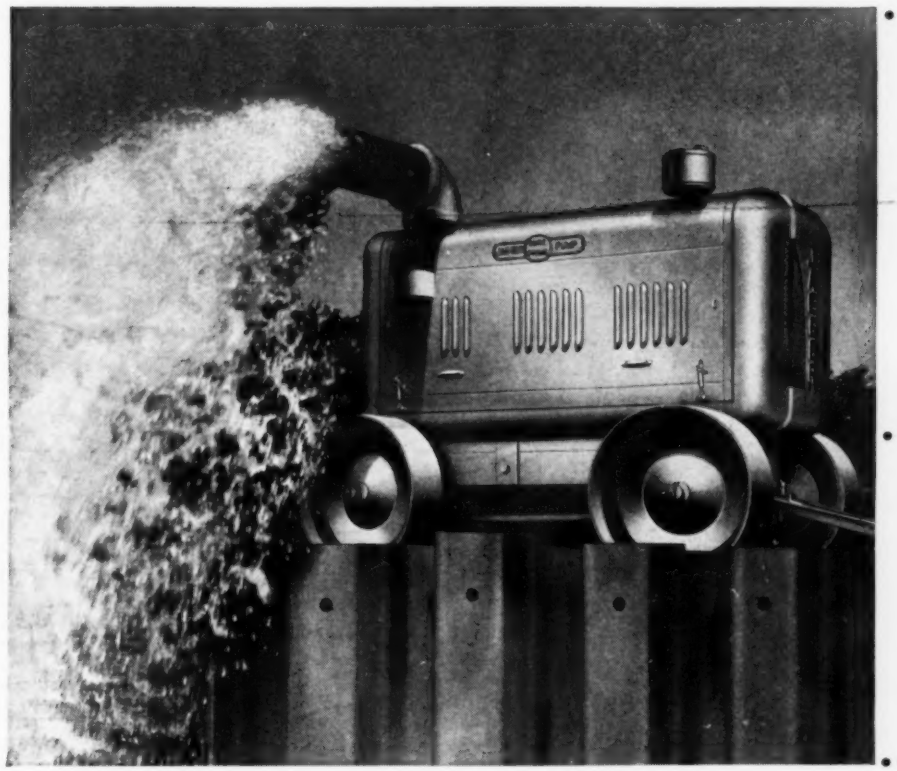
Write for Bulletin No. 100 which gives further information on Conkey Sludge Filters.

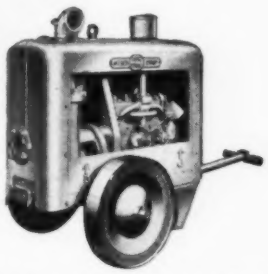
General American
TRANSPORTATION CORPORATION
process equipment • steel and alloy plate fabrication
SALES OFFICE: 10 East 49th St., Dept. 830c, New York 17, N. Y.
WORKS: Sharon, Pa., East Chicago, Ind.
OFFICES: Chicago, Sharon, Louisville, Orlando, Washington, D. C. Pittsburgh, St. Louis, Salt Lake City, Cleveland.

PROCESS EQUIPMENT DIVISION GENERAL AMERICAN TRADE MARK

JAEGER "SURE PRIME" PUMPS

"Bantam" aluminum pump weighs only 55 lbs. with engine —delivers 3000 gallons an hour —the champion lightweight.



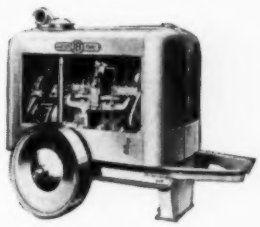


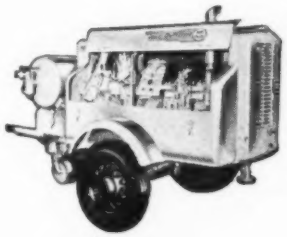
Enclosed-Type—Capacities up to 240,000 gph—Pressures up to 275 lbs.

Extra-powered, conservatively rated and sure-primed by two separate, simultaneous actions, Jaeger pumps are built for high efficiency and heavy duty service. Overall enclosures, exclusive with Jaeger, insure continued top-performance with dry, quick-starting, smooth-running engines in all weather.

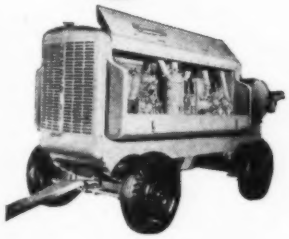
Close-grained shell castings designed for high pressures, long-life Lubri-Seal always accessible for inspection, and replaceable liners or seal rings mean extra years of low-cost service.

Gasoline and electric powered dewatering pumps are available in 1½" to 10" sizes, including caisson and diaphragm pumps. Also compact, portable 2", 3" and 4" pressure pumps. Jaeger 4" high pressure pump, shown at right, is self-priming and delivers water at up to 275 lbs. pressure for emergency supply work, jetting, fire-fighting, etc. Send for complete Catalog.





60, 105 and 160 ft. trailers



210, 315, 500 and 600 ft. portables on 4 wheels

JAEGER "AIR PLUS" COMPRESSORS

60 TO 600 CFM PORTABLES — GAS, DIESEL, ELECTRIC POWER

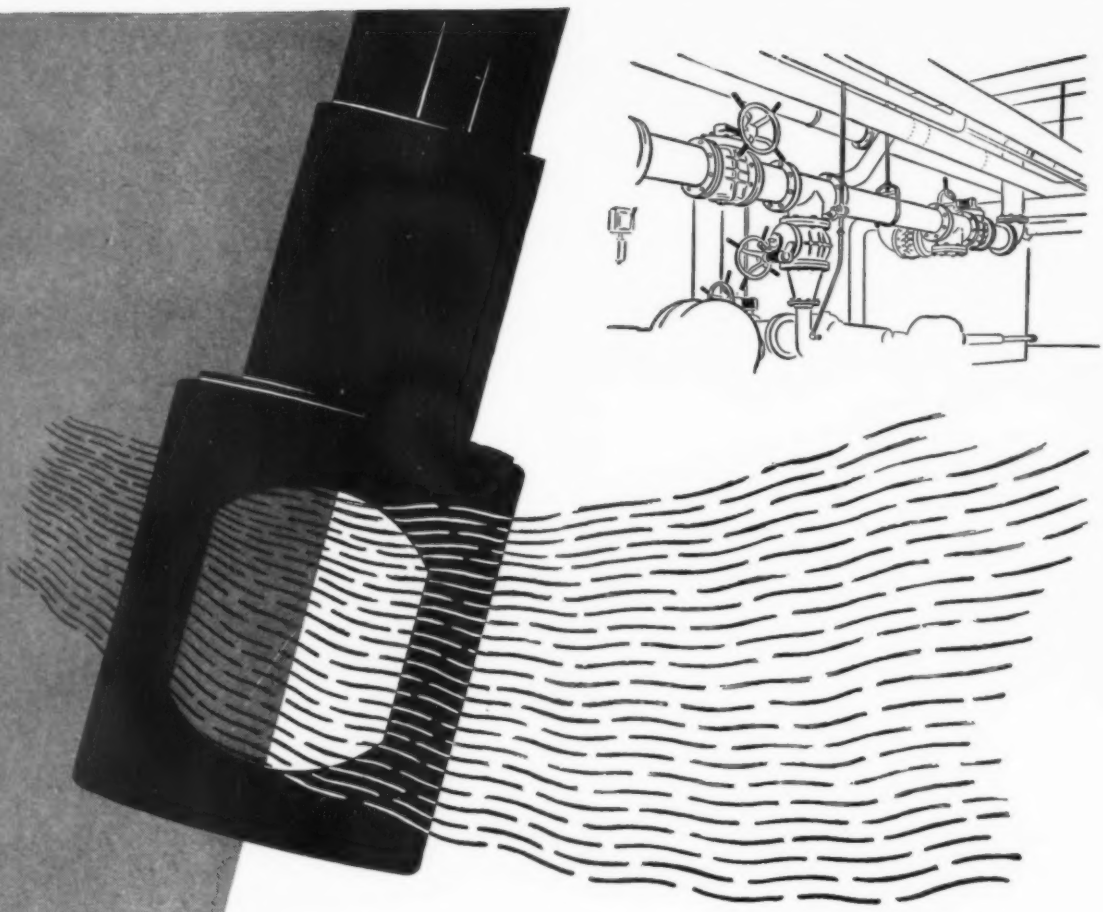
Designed for steadier pressure, with high-efficiency compressor units and up to 70% larger air receivers, "AIR PLUS" Compressors maintain the 90-100 lbs. behind your tools that keeps them working at full power and speed.

Compressors and engines are build to the same micro-precision standards. Valves are 75% to 100% larger for free air flow, and "ultra lapped" to leak-proof closure. Patented "Fuel Miser" engine speed control saves up to 32% of fuel. Engineered throughout to insure a dependable supply of air, with force-feed lubrication, lifetime clutches and bearings, sectional radiators and intercoolers, Timken bearing trailer wheels or truck or skid mounting. Send for complete Catalog.

Distributors in 130 Cities Sell, Service Jaeger Pumps, Compressors, Mixers, Hoists

THE JAEGER MACHINE COMPANY — Main Office and Factory COLUMBUS 16, OHIO

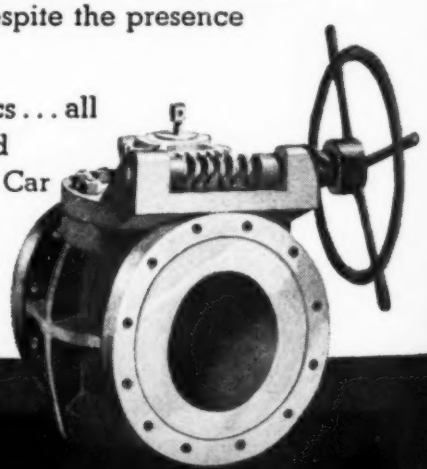
Regional Offices: 1504 Widener Bldg., PHILADELPHIA 7 — 226 N. La Salle St., CHICAGO 1— 235 American Life Bldg., BIRMINGHAM 1



Easy Flow for Sludge Lines

Freedom from possible obstruction is the paramount operating advantage of the full-pipe-area A.C.F. Lubricated Plug Valve. The exclusive cylindrical plug design has no pockets and permits uninterrupted flow through the valve despite the presence of rags or other foreign material.

Raw sewage, sludge, air, gas, water caustics . . . all are efficiently handled by A.C.F. Lubricated Plug Valves. Ask for catalog 3(WS). American Car and Foundry Company, Valve Dept., 30 Church St., New York 8, N. Y.



American Car and Foundry Co.

LUBRICATED
PLUG VALVES

THE FIRST

SLUDGE INCINERATION SIMPLIFIES PLANT LAYOUT . . .

incinerator to be

by any

PROVIDES SLUDGE DISPOSAL IN ONE FULLY AUTOMATIC OPERATION . . .

since the war

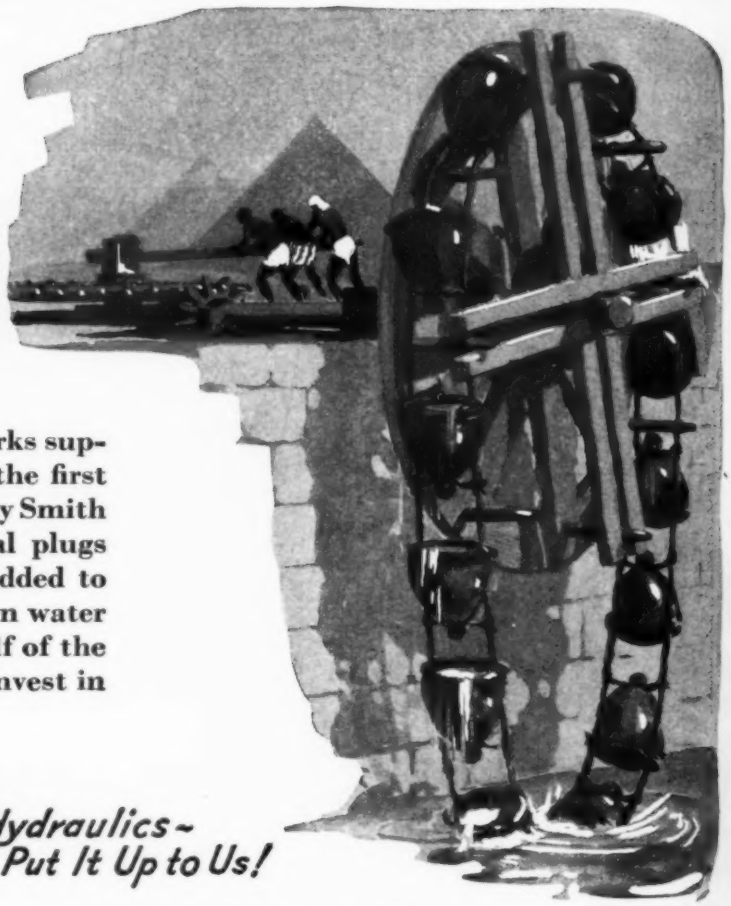
ELIMINATES UNSIGHTLY DRYING BEDS, SLUDGE DUMPS, LAGOONS .

by the city of

Massachusetts, to

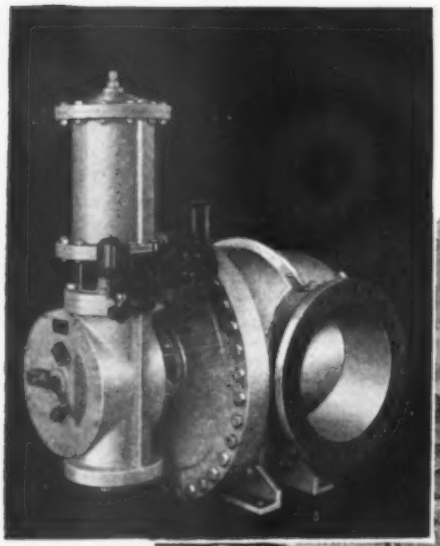
60 Wall Tower, New York 5, N.Y. In Canada: University Tower Bldg., Montreal

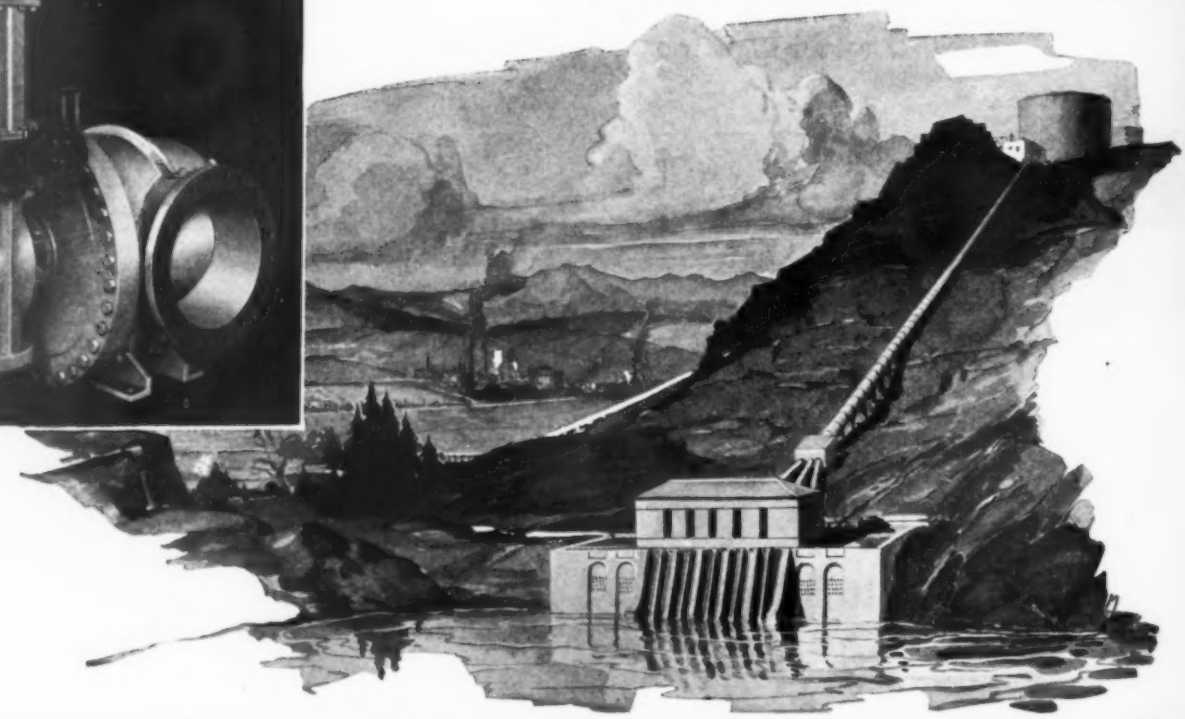
Modern WATERWORKS *possible with* Rotovalves *by* SMITH



MOVING ever onward, the huge water works supply systems of today have evolved from the first crude inventions of antiquity! Rotovalves by Smith with their full circular openings, conical plugs and electrically welded seat rings have added to the efficiency and reliability of the modern water works and sewage systems. Assure yourself of the utmost return on your Valve Dollar and invest in Rotovalves. Write for our bulletin.

If It's Hydraulics~ Put It Up to Us!



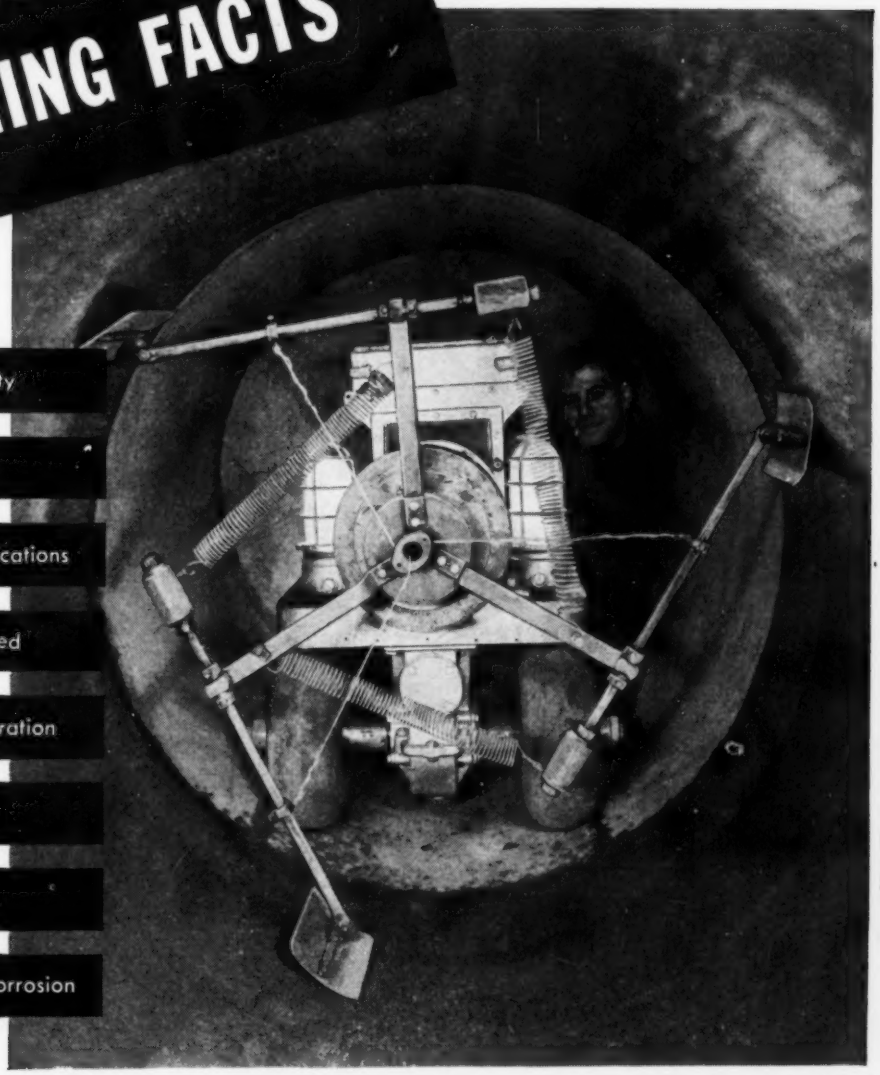


S. MORGAN SMITH Co.

YORK, PENNA. U.S.A.

CENTRILINE
PIPE LINING FACTS

▶▶▶▶ WRITE TODAY
FOR THE NEW
CENTRILINE BULLETIN



1. Permanently increases carrying capacity
2. Reduces pumping costs
3. Conforms strictly with A.W.W.A. specifications
4. Varies mortar-thickness according to need
5. Prevents leakage due to exterior penetration
6. The best protection for new steel mains
7. Reclaims abandoned mains
8. Affords permanent protection against corrosion

If mains are losing their efficiency or if new mains are planned you should consider Centrilining. Our new bulletin will tell you the many advantages of the Centriline Process . . . better still, we suggest you consult our competent staff of hydraulic engineers.

CENTRILINE CORPORATION

148 CEDAR STREET • NEW YORK 6, N. Y.

CEMENT MORTAR LININGS FOR WATER MAINS
CENTRIFUGALLY APPLIED
in strict conformity
with A.W.W.A. Specifications

Baltimore's back steps will be cleaner, too!

THE PEOPLE of Baltimore have long been noted for the pride they take in their doorsteps. Street-long rows of sparkling white steps are a Baltimore trade-mark!

But what of Baltimore's back steps? Is the day far distant when a great percentage of the garbage cans—breeders of filth and disease—will have disappeared from this proud city's back yards?

Farsighted officials have recently taken steps that will permit the installation of General Electric Disposalls* in Baltimore homes!

They have revised Baltimore's regulations after carefully studying every aspect of the new, revolutionary kitchen appliance that shreds all food waste and flushes it into the sewer.

They had to be convinced that General Electric had developed a mechanical food-waste disposal process *that* (1) provided a mechanism correctly engineered and manufactured for long and dependable service; (2) properly shredded food wastes, and homogenized and congealed greases; (3) effectively flushed these wastes through home plumbing and public sewers without danger of clogging these vital conduits; (4) mixed these water-buoyant wastes with sewage, into a liquid waste that could be readily treated at the city's sewage treatment plant.

These men found all the answers in the Disposall method. They gave the "go-ahead" to a development

that leads to a cleaner, more convenient, more healthful future for every Baltimorean!

You, too, will recognize the Disposall method as one of the outstanding sanitation achievements of the 20th century, once you've examined it closely. The questions and answers below are mere preliminary ones. We hope, for the sake of your community, they'll prompt you to inquire further into this method which is proving itself in hundreds of communities all over the land!

ANSWERING YOUR QUESTIONS ABOUT THE *DISPOSALL* METHOD OF ELIMINATING GARBAGE

1. "Why should food waste be instantly removed?"

Because, like human waste, food waste is organic matter —odorous and putrefactive. Attracting germ-spreading flies and rodents, it is a dangerous source of infection.

No less a threat to community health than was the outhouse, the garbage can should be as completely eliminated. The Disposall method can mean a garbage-free community!

2. "Has the Disposall method been adequately tested?"

Yes! Disposalls have been installed in several hundred communities. Their efficiency in actual use, plus extensive laboratory testing for more than ten years, has proved Disposalls to be an efficient method for the removal of food waste and the elimination of the home garbage can.

3. "Is there any danger of sewer stoppages?"

No! The Disposall shreds food wastes into a form of liquid waste that is more successfully carried through sewers than ordinary sewage solids.

Although similar to human wastes and other solids now flowing through sewer lines, Disposall processed food wastes are smaller, more water-buoyant and uniform in character. These wastes are ground in a controlled amount of cold water, which congeals and homogenizes greases and flushes these wastes away from the home.

4. "Will we have to expand our plant's capacity?"

For many years the number of Disposalls in a community may not become large enough to add appreciably to the loading on the normal treatment plant. Even if every home in your community used a Disposall, the *volume* of sewage flow would be increased by *less than two per cent.*

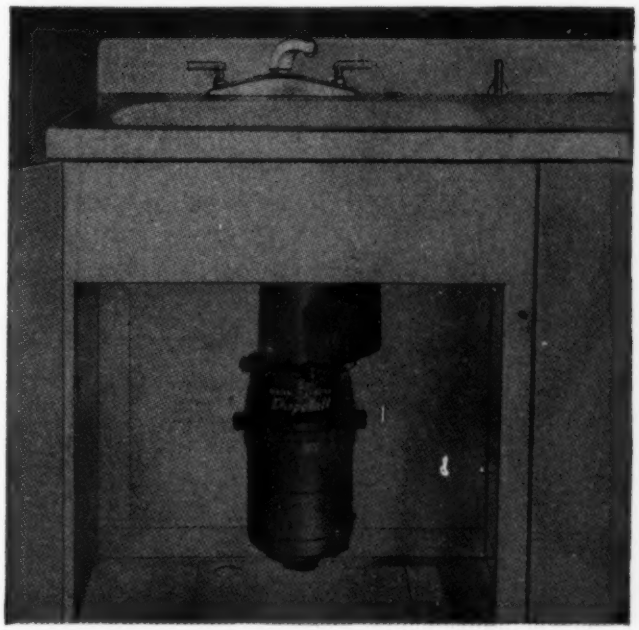
The addition of food wastes to the sewage flow will naturally increase solids and organic loading on the plant. Many plants will be able to handle this increased loading for some time; others can be expanded when necessary.

5. "Are sewage treatment plants necessary?"

Whether or not Disposalls are used, we believe that every community should have a sewage treatment plant. The Disposall method accentuates the need for these plants. They are needed! They provide a vital safeguard for public health!

6. "How can I get more information?"

Further information about Disposalls—and the Disposall method—is yours for the asking. We shall be glad to send complete information, to answer any specific questions you may want to ask. Please write to Department WSW-484, General Electric Company, Bridgeport 2, Connecticut.



The General Electric Disposall shown installed in the Electric Sink. Food waste is scraped into drain opening and safety cover is placed in position. Cold water tap is then turned on, starting the Disposall operation. Food waste is shredded and flushed away automatically.

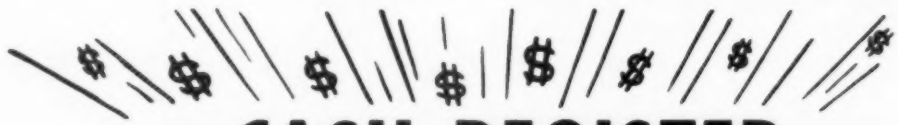
Disposall

*General Electric's registered trade-mark for its food-waste disposal appliance.

CONVERTS FOOD WASTE TO SEWAGE—ELIMINATES "GARBAGE"

GENERAL ELECTRIC

Accurate METERING

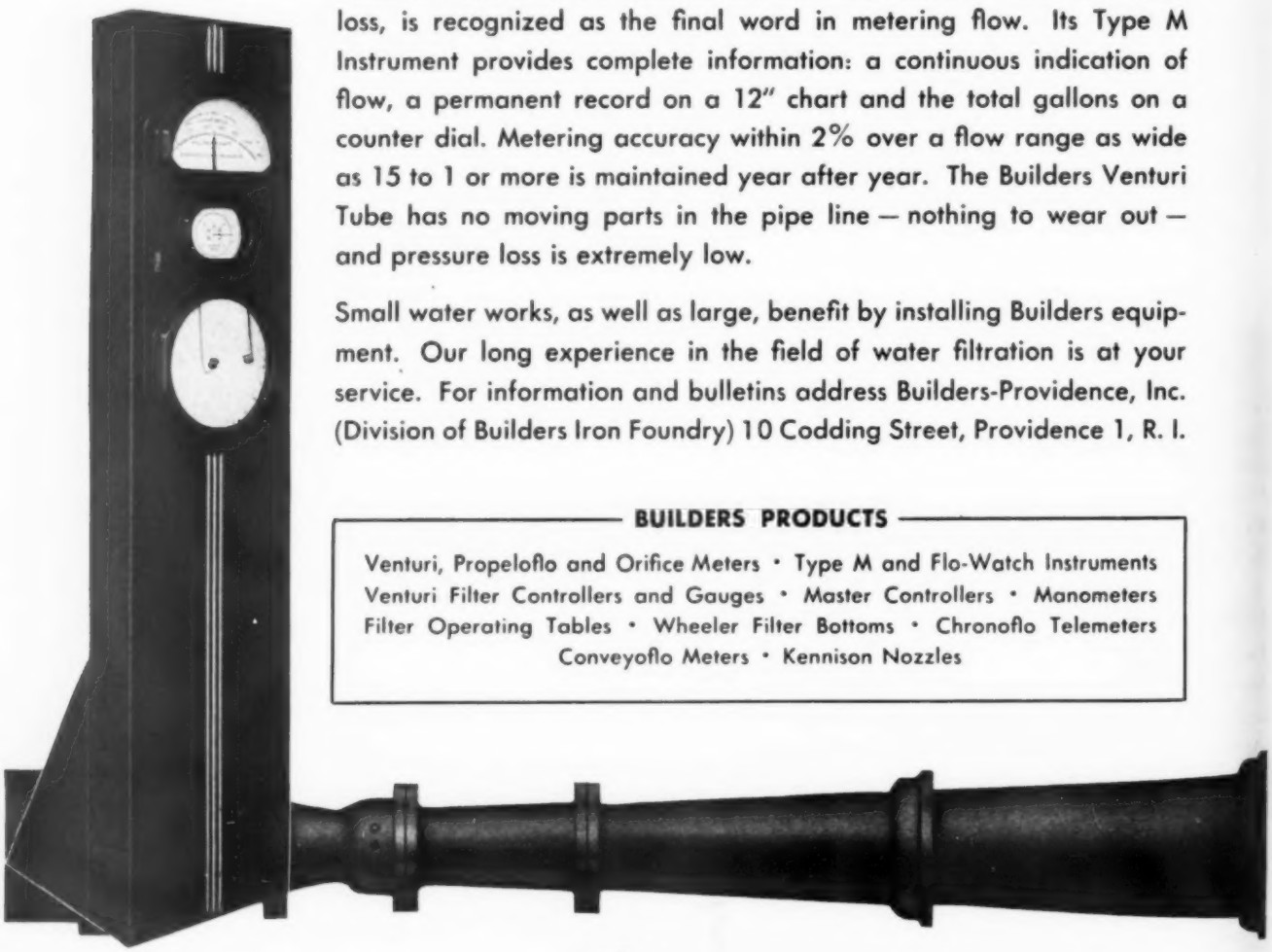


THE CASH REGISTER

OF YOUR WATERWORKS

Water is *money* in any filtration plant — large or small. Pumping water to waste costs as much as delivering it to customers, but adds nothing to income. Accurate metering detects those unseen leaks, saving possibly millions of gallons of water and greatly reducing operating costs.

Builders Type M Register-Indicator-Recorder and Herschel Standard Venturi Tube.



Builders Venturi Meter, famous for accuracy, long life and low head loss, is recognized as the final word in metering flow. Its Type M Instrument provides complete information: a continuous indication of flow, a permanent record on a 12" chart and the total gallons on a counter dial. Metering accuracy within 2% over a flow range as wide as 15 to 1 or more is maintained year after year. The Builders Venturi Tube has no moving parts in the pipe line — nothing to wear out — and pressure loss is extremely low.

Small water works, as well as large, benefit by installing Builders equipment. Our long experience in the field of water filtration is at your service. For information and bulletins address Builders-Providence, Inc. (Division of Builders Iron Foundry) 10 Codding Street, Providence 1, R. I.

BUILDERS PRODUCTS

Venturi, Propeloflo and Orifice Meters • Type M and Flo-Watch Instruments
Venturi Filter Controllers and Gauges • Master Controllers • Manometers
Filter Operating Tables • Wheeler Filter Bottoms • Chronoflo Telemeters
Conveyoflo Meters • Kennison Nozzles

BUILDERS PROVIDENCE

Instruments

Sleepy Eye is wide awake!

. . . when it comes to furnishing its residents with a *dependable* supply of water

Adequate storage capacity in elevated steel tanks is evidence of sound planning on the part of a water works engineer. In fact, many engineers consider the installation of an elevated water storage tank the most important step in modernizing a municipal water supply system. The 300,000-gal. Horton tank shown at the left was recently erected at Sleepy Eye, Minn., to maintain gravity pressure in the distribution system serving that city. Some of the benefits that the residents of Sleepy Eye may expect from their new tank are:

1. More uniform pressures throughout the distribution system
2. More dependable supply of water for fire protection
3. Lower insurance rating for all property in the city
4. Maintenance of pressure during temporary power failure
5. Reduced pumping costs

Engineering data on water system at Sleepy Eye

Source of Water—Wells.

Number of Customers—960 (including 3 major industrial plants).

Pumping Equipment—Three motor-driven turbine pumps of 200 gpm, 300 gpm and 400 gpm capacity.

Distribution Mains—9 miles of 6-inch and 4-inch mains.

Transmission Mains—1 mile of 10-inch and 8-inch mains.

Storage Capacity—300,000 gallons in elevated tank.

Maximum Consumption Rate—400 gpm.

Minimum Consumption Rate—100 gpm.

CHICAGO BRIDGE & IRON COMPANY

Chicago 4.....2198 McCormick Bldg.
New York 6.....3390-165 Broadway Bldg.
Cleveland 15.....2262 Guildhall Bldg.
Los Angeles 14.....1455 Wm. Fox Bldg.
Birmingham 1.....1586 North 50th St.
Atlanta 3.....2181 Healey Bldg.
Plants in Birmingham, Chicago, Salt Lake City and Greenville, Pa.

Tulsa 3.....1646 Hunt Bldg.
Houston 1.....5615 Clinton Dr.
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Havana.....402 Abreu Bldg.
San Francisco 11.....1283-22 Battery Street Bldg.
Detroit 26.....1551 Lafayette Bldg.
In Canada: Horton Steel Works, Limited, Fort Erie, Ont.

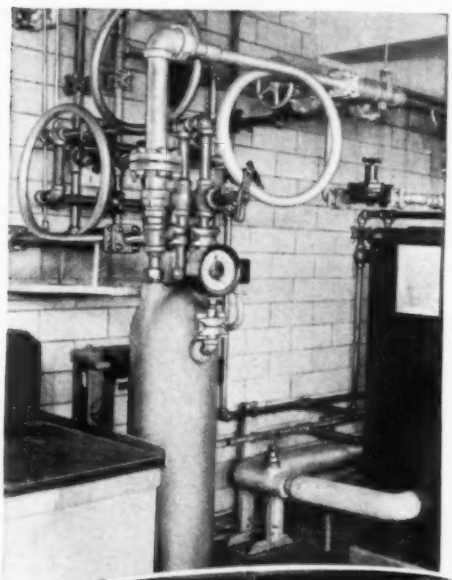
In milltown or metropolis Crane valves meet the need

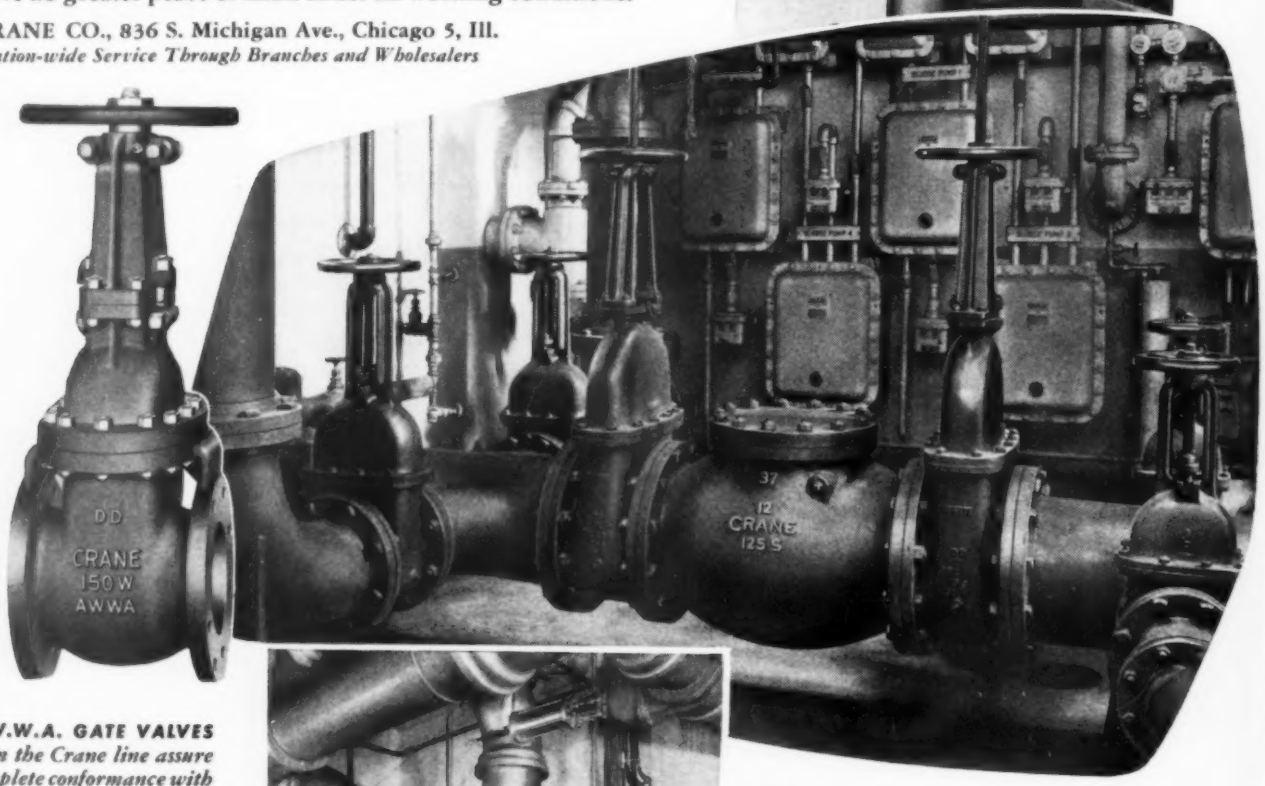
In public utility services regardless of size, what is needed more than the assurance of dependable operation? For water and sewage systems, where the nation's health is at stake, what is more necessary than dependable flow control?

In valves as in fittings, Crane Quality meets today's highest standards for water supply and sewage disposal practices. The community that builds with Crane equipment in pumping station, treatment plants, and streets, can have no greater assurance of maximum value and dependability in piping materials. The men who design and operate the systems can have no greater peace of mind under all working conditions.

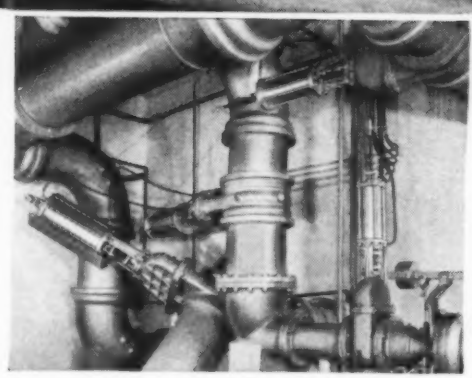
CRANE CO., 836 S. Michigan Ave., Chicago 5, Ill.
Nation-wide Service Through Branches and Wholesalers

CHLORINE EVAPORATOR SYSTEM *for sewage treatment—safe and dependable because piping materials are Crane Quality.*





A.W.W.A. GATE VALVES *from the Crane line assure complete conformance with latest specifications, plus unsurpassed performance for valves in this class. Shown here is pattern No. 2495, double-disc, OS&Y, with flanged ends. Made in sizes 2 to 24 in. See your Crane Catalog supplement 3A, page 126.*



SLUDGE INLET LINES *to digester in sewage plant featuring Crane iron gate and check valves, and fittings.*

CYLINDER-OPERATED *Crane iron gate valves in filter lines in large water treating plant. Crane supplies all types of remote controls for valves.*

VALVES · FITTINGS · PIPE

CRANE

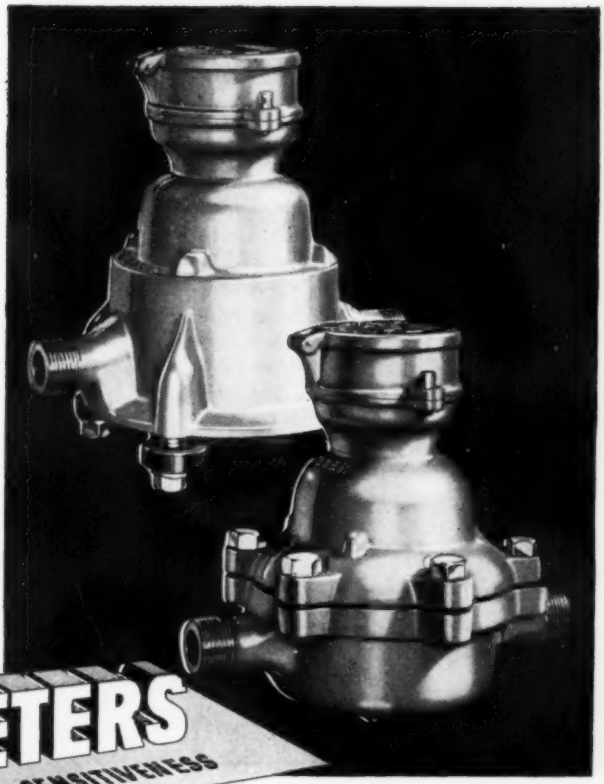
PLUMBING AND HEATING

"Condition of meter deciding factor in award of our business to Badger"

"The excellent condition of this meter, examined after 12 years of service, was one of the deciding factors in the award of our business to your company" wrote a prominent waterworks official.

There's a Badger Meter size and type for every waterworks service. More than 3-million satisfied users in over 5,000 communities prove "It pays to BUY BADGER."

BADGER METER MFG. CO., MILWAUKEE 10, WIS.



Every water works superintendent should have a copy of our chart:
"RELATIVE CARRYING CAPACITIES OF PIPE LINES", No. 668
Send for yours today (no charge)

BADGER Water METERS

ACCURACY · LOW COST MAINTENANCE · DURABILITY · SENSITIVENESS

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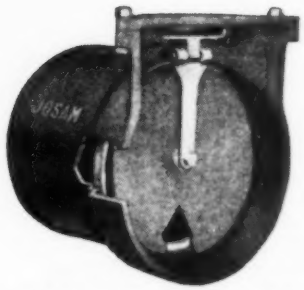
PROTECTION *for* PIPE LINES

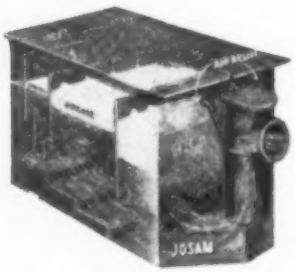
SEWER VALVE Series 1170

Flap is normally open 3/4" to allow continuous circulation. Can be adjusted to closed seat type if desired. Inside of valve is offset to prevent fouling. Flap closes tightly with slightest backflow, preventing entry of any backwater or sewage through the drain lines. Supersensitive side swing flap opens wide to permit full water way up to size of pipe for which it was designed.



FLAP VALVE Series 1110½

When placed at the discharge end of sewer pipes, the Josam Flap Valve is an absolute safeguard against the backflow of sewage. In sewer lines it can be placed at the manhole. It will also serve as a sluice, river or tide water outlet and can be used between settling basins in sewage disposal plants.



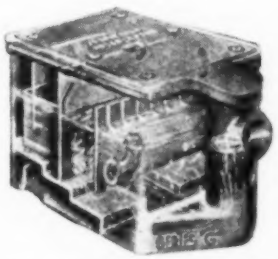
GREASE INTERCEPTOR

The Josam Cascade Grease Interceptor provides maximum grease interception and retention for all types of installations. Grease is removed from waste water before it clogs drain lines. Made in types and sizes for any requirement. Equipped with flow control to insure maximum grease retention efficiency.

SHOCK ABSORBERS for Water Supply Pipe Lines

To eliminate disturbing noises and overcome destructive pounding of water hammer in pipe lines. Especially desirable where quiet is essential. Prevents breakdown of pumps, valves, connections, machinery and equipment.

Josam PRODUCTS STANDARD OF AMERICA ESTABLISHED 1914



GAS-OIL INTERCEPTOR

The Josam Gas-Oil Interceptor separates dangerous volatile liquids from waste water or drainage systems, and safely ejects them. Prevents fire, explosion and water pollution. Equipped with flow control to provide maximum interception efficiency. Made in a wide range of types and sizes.

WRITE FOR INFORMATION AND FREE LITERATURE ON THESE JOSAM PRODUCTS

Original Double Drainage Floor, Roof and Shower Drains • Genuine Non-Clog Triple Drainage Floor Drains
Backwater Sewer Valves • Interceptors • Shock Absorbers for Pipe Lines • Moderator
Mixing Valves for Showers • Swimming Pool Equipment

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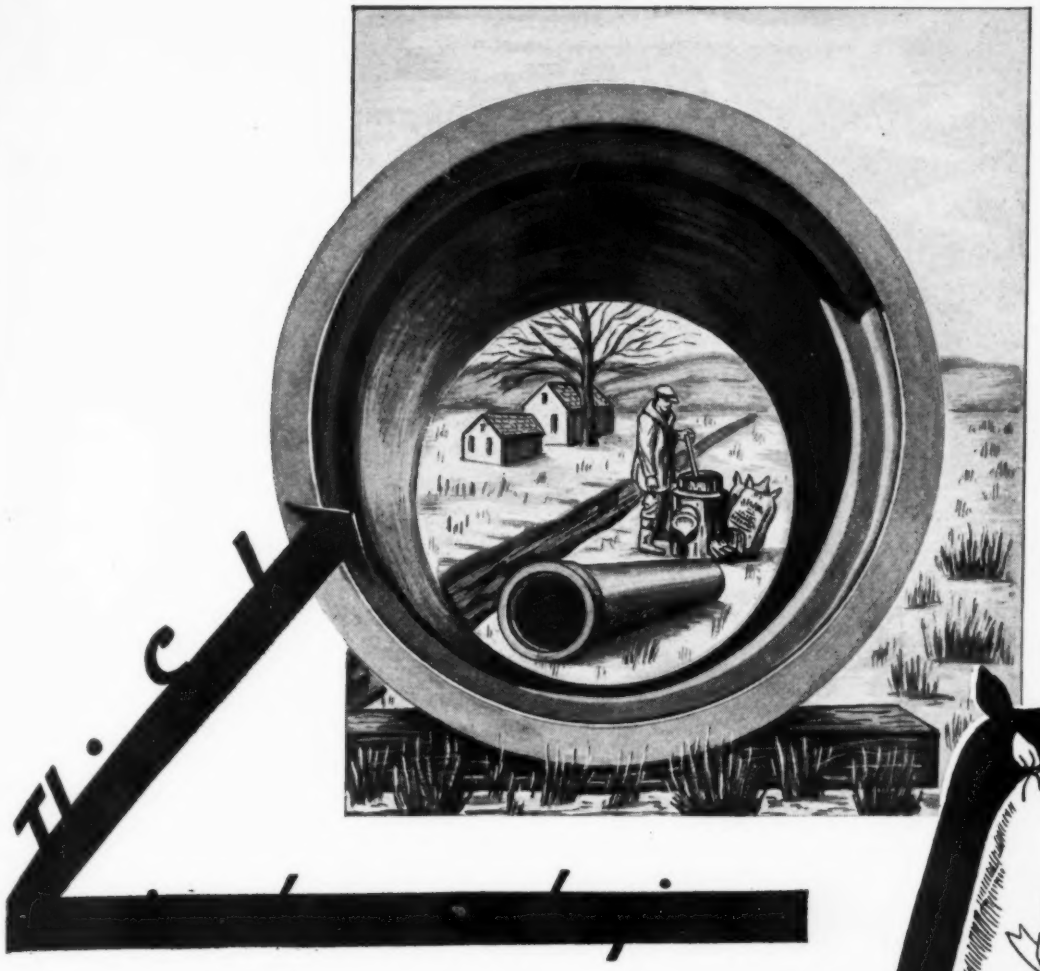
THERE ARE NO SUBSTITUTES FOR JOSAM PRODUCTS



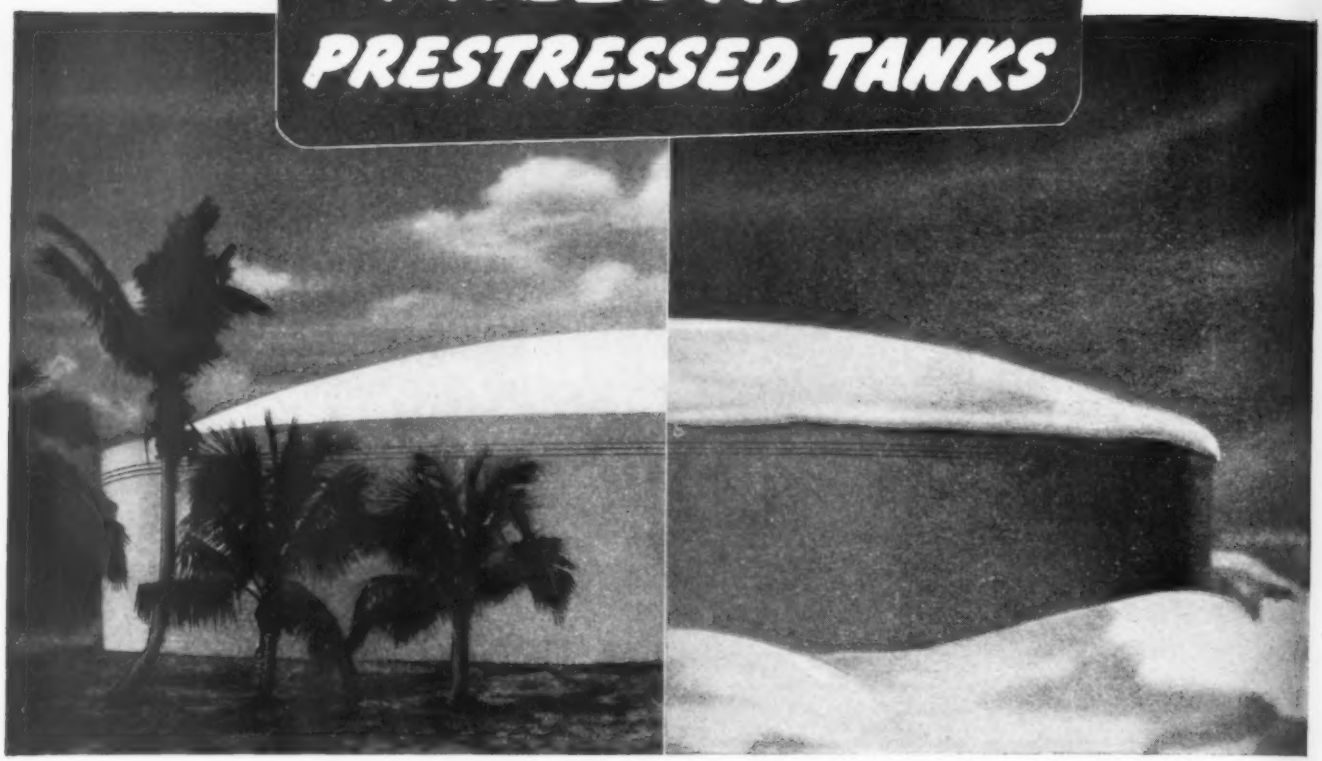
DOWN through the years the development of cast iron bell and spigot pipe has resulted in a product that can unquestionably last for a century and more. The jointing compound used should naturally be a matter of deep concern. HYDROTITE, time tested and proven for over 30 years, is the one jointing compound you know you can rely on. Self-caulking, self-sealing HYDROTITE costs less to buy and use, requires minimum of unskilled labor, makes tighter joints and withstands the effects of vibration and time. Used with FIBREX, a bacteria free, easy to use packing, it makes an unbeatable water tight joint.

Always use FIBREX, the sanitary, bacteria-free packing that costs about 30% less than braided jute. Sample on request.

HYDRAULIC DEVELOPMENT CORP.

50 CHURCH STREET, NEW YORK, N.Y.
Plant at WEST MEDFORD, BOSTON, MASS.

PRELOAD PRESTRESSED TANKS



Eliminate Long Term Maintenance Expense By Using PRELOAD Wire Wound Prestressed Concrete Tanks

Internal corrosion, and external deterioration due to severe weather conditions, are the principal causes of repair and rehabilitation jobs on metal tanks or tanks designed and built by conventional methods of reinforced concrete construction.

PRELOAD PRESTRESSED CONSTRUCTION, utilizing a homogenous, smooth, dense interior and exterior gunite surface, offers no exposed metallic areas where corrosive action can take place. The balanced stresses attained by carrying the load with PRELOAD PRESTRESSING cannot cause cracks due to internal strain. The exterior finish is proof against spalling and other weather hazards.

In the summer heat of Miami or the winter cold of Halifax, from the torrid heat of Colombia to the extreme cold of Canada, PRELOAD TANKS have long demonstrated their ability to stand up under any weather conditions.

Built with locally available materials and labor, and adaptable to creative design techniques which will fit into residential environments, PRELOAD TANKS offer substantial economies in first cost and long-term maintenance costs. Specify PRELOAD TANKS for municipal or for industrial water storage; storage of oil and other bulk liquids; and for use in sewage disposal plants.

Send for Bulletin

THE PRELOAD COMPANIES

420 LEXINGTON AVE., NEW YORK, N. Y.

Designers and Builders of Prestressed Concrete Storage Tanks and Pressure Pipe
Prestressed Concrete Bridges and Cement Lining of Pipe in Place

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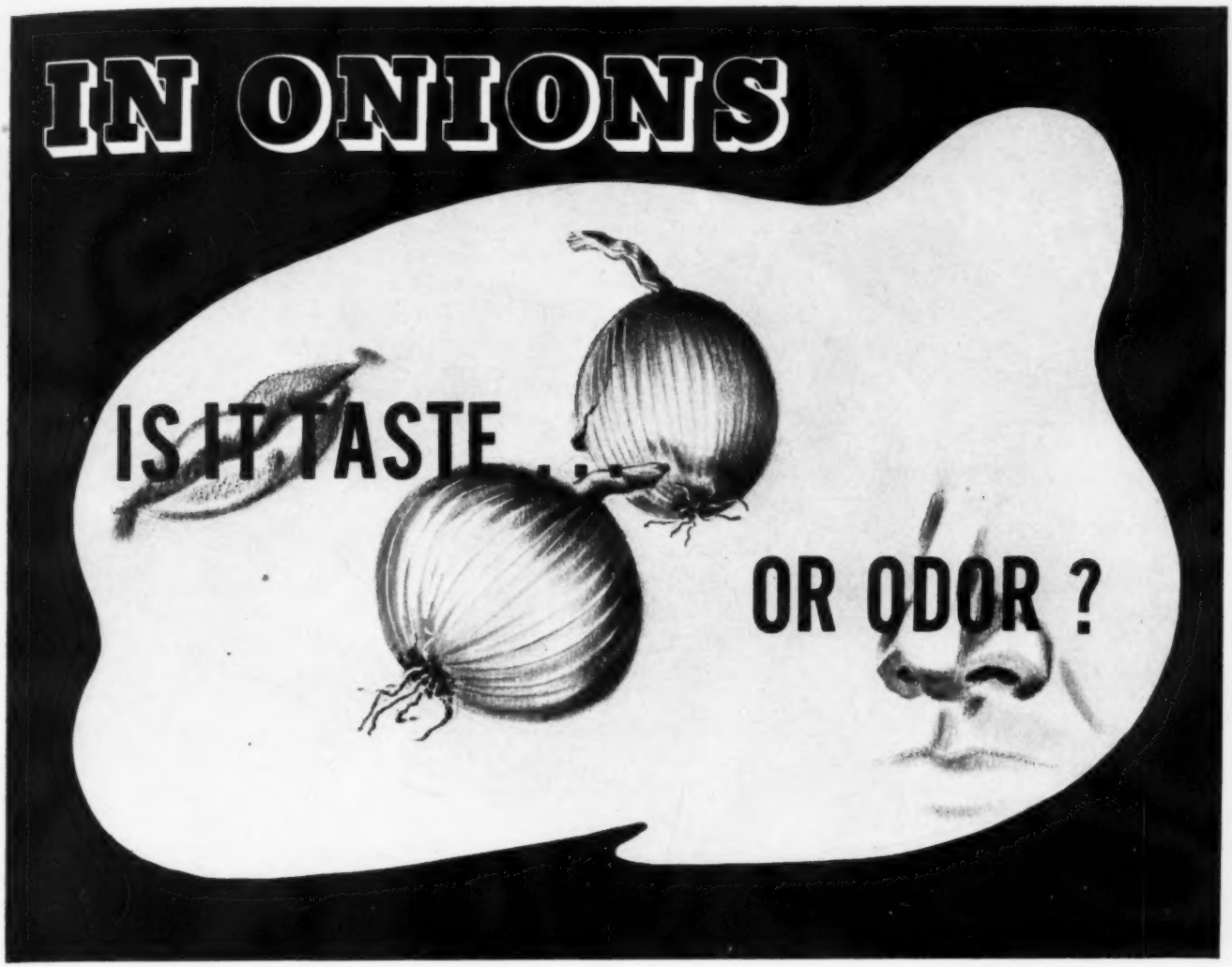

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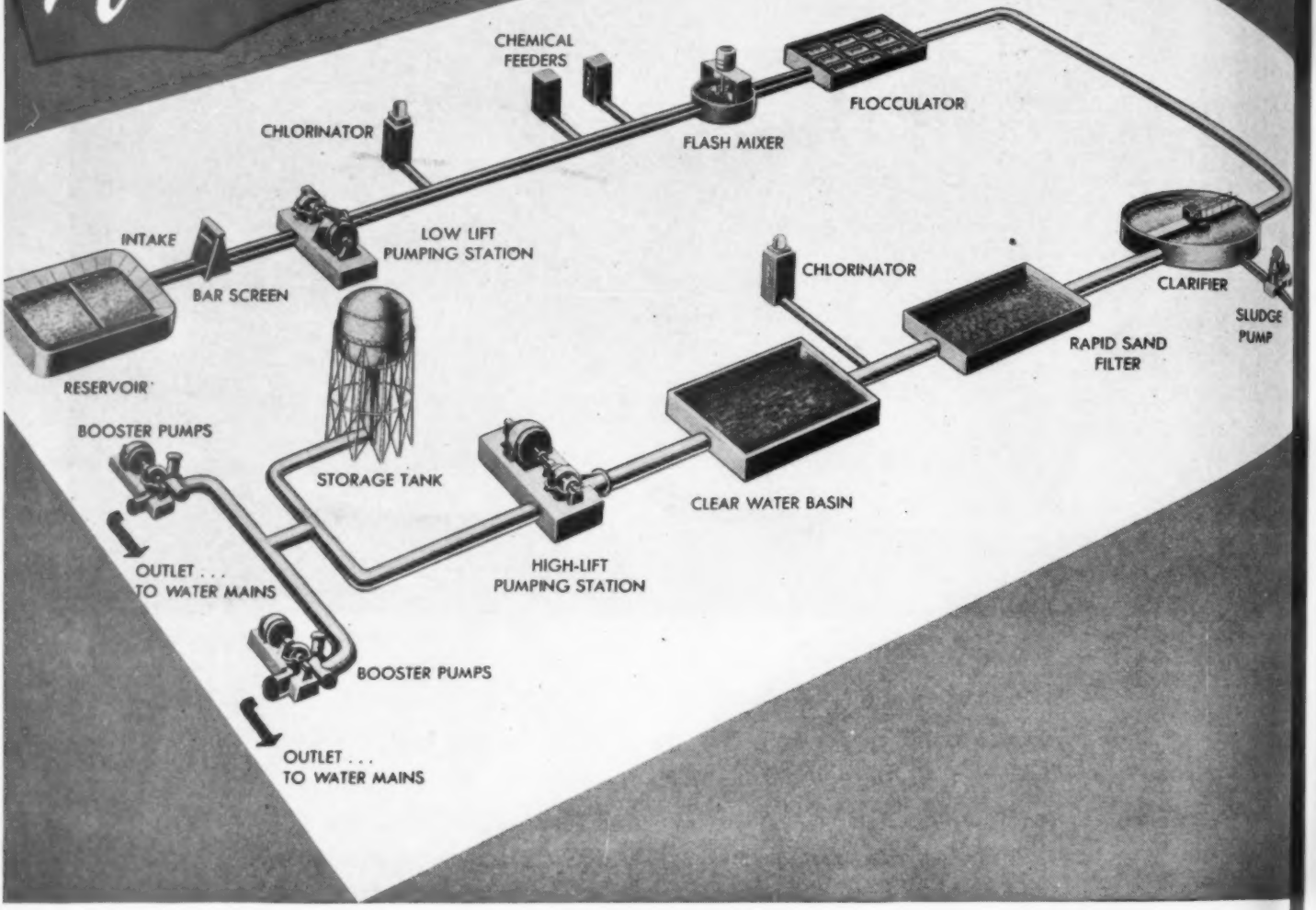

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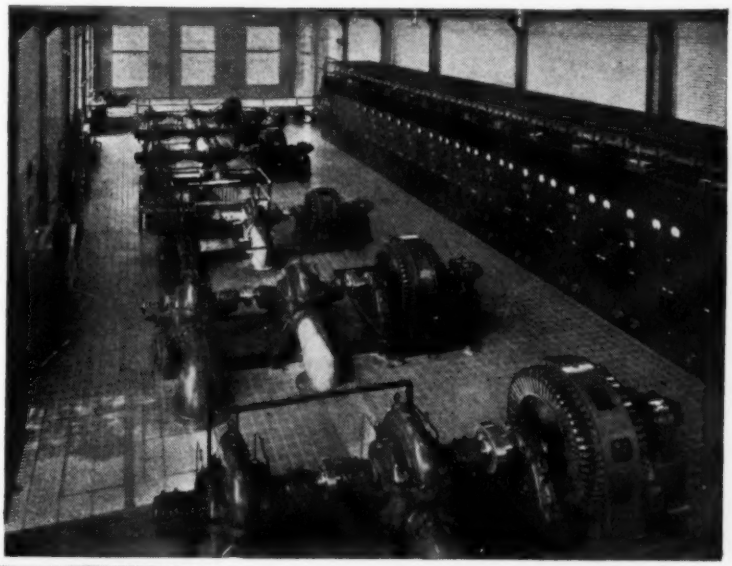

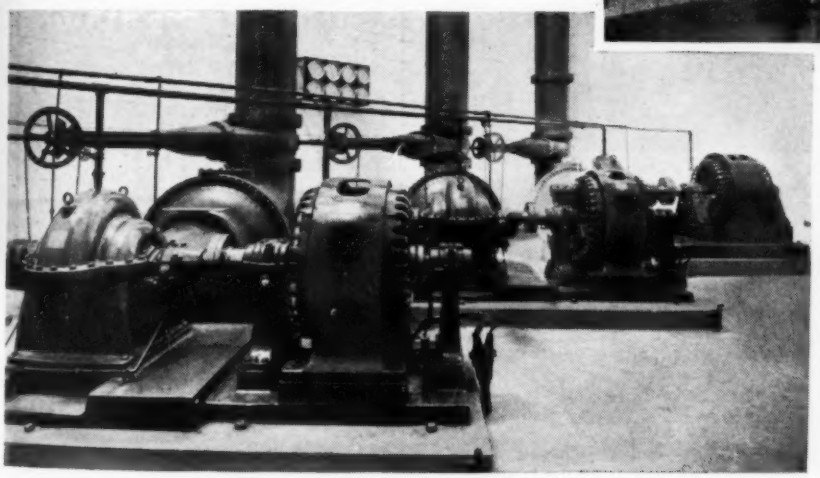

LEFT: Basement floor, showing three of four 500-hp Westinghouse motors, each of which drive a 30 mgd raw-water pump. Three 900-hp Westinghouse vertical motors, driving clear-water pumps, are also located on this floor but are not seen in this picture.

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—SAFETY—SERVICE—SATISFACTION—

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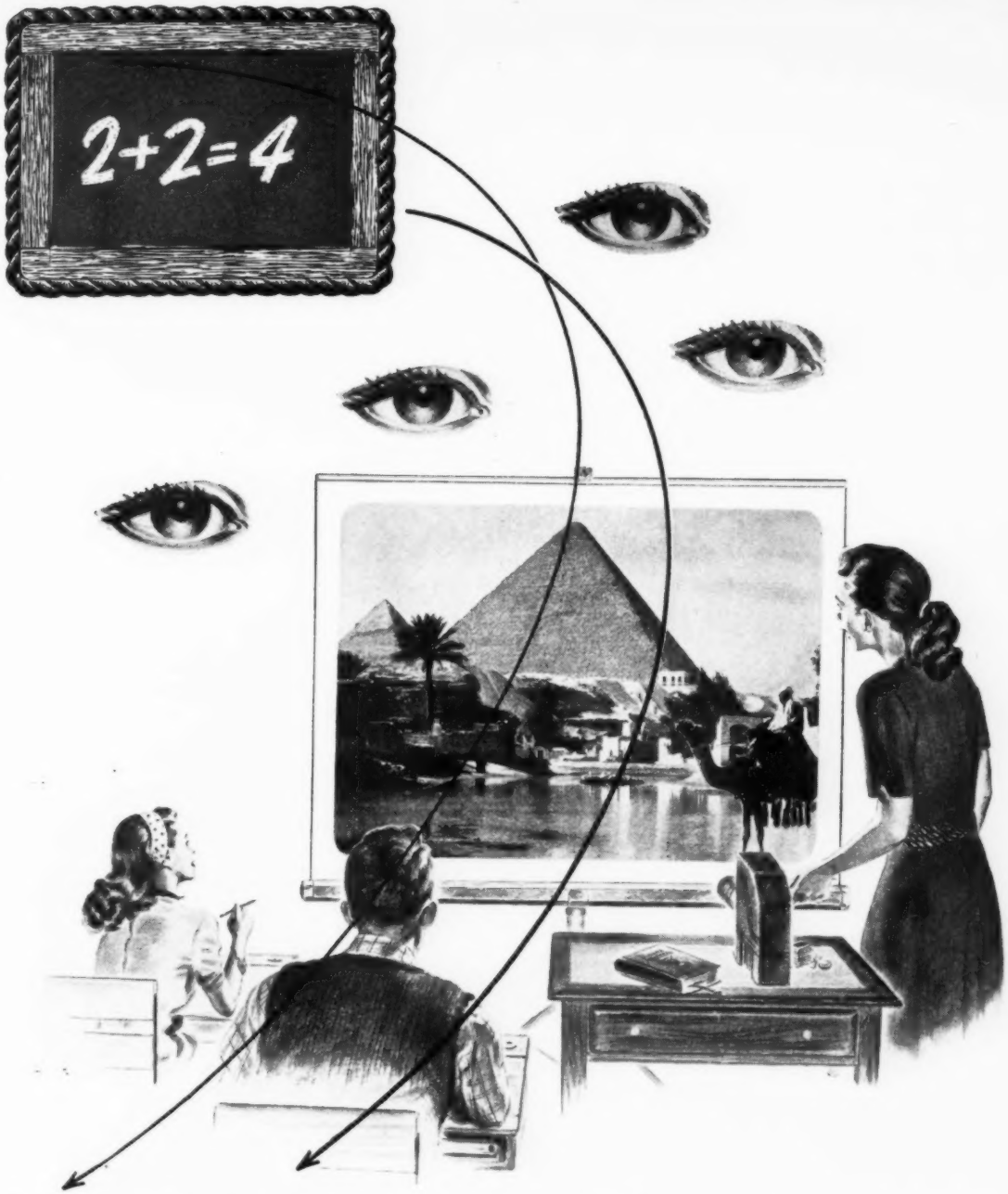

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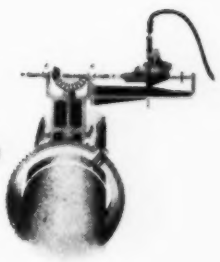

pipe cutting machine

new 18" cleaning head

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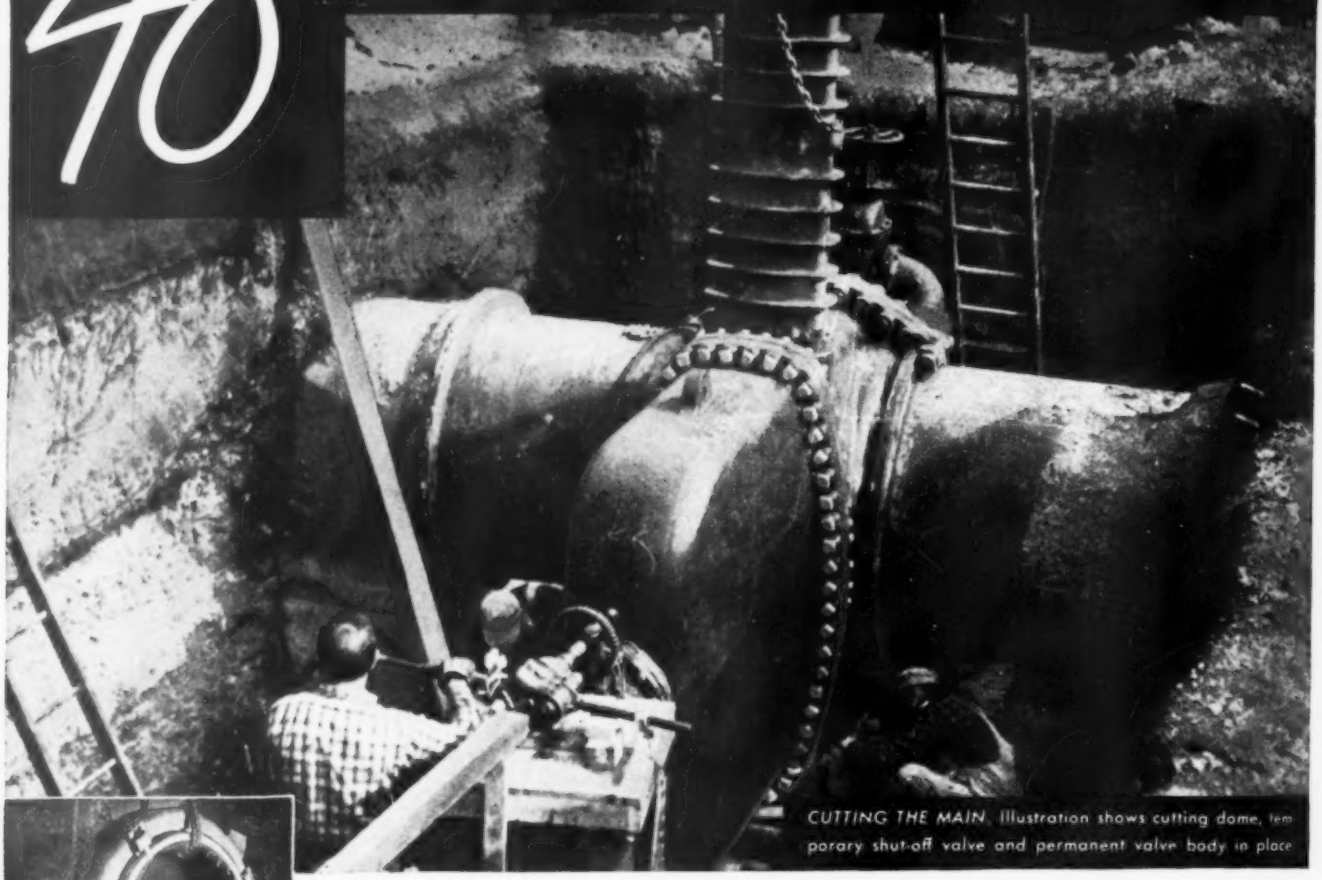
*48" GATE VALVE INSERTED UNDER PRESSURE

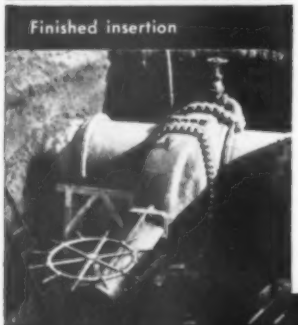
to Avoid Stoppage of Steel Production



CUTTING THE MAIN. Illustration shows cutting dome, temporary shut-off valve and permanent valve body in place

Section removed from main

Finished insertion



At a large Eastern steel mill uninterrupted production depended upon a thirty year old, 48" cast iron main, carrying sea water used for cooling the furnace jackets. The main was in poor condition — badly corroded and electrolytically pitted.

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*First 48" insertion ever made under pressure. One month later a second 48" gate valve was inserted by Smith for the Water Bureau, Philadelphia, Pa.

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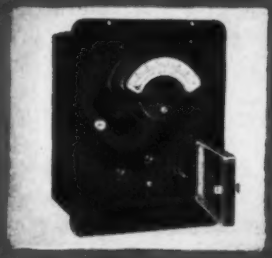

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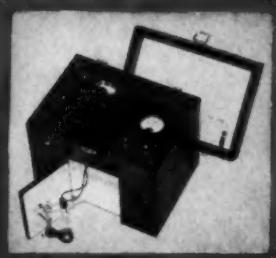

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WATER & SEWAGE WORKS

VOL. 95, No. 4 APRIL, 1948

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USE OF REYNOLDS NUMBER— FACT OR FANCY?

By CHAS. H. CAPEN
Chief Engineer
NO. JERSEY DIST. WATER SUP. COMM.
WANAQUE, N.J.

NUMEROUS inquiries have developed the general proposition that most college students who are exposed to the study of hydraulics are entirely at sea regarding pipe flow formulas. The vagueness of this understanding increases greatly as time elapses after the course or courses are completed. It is little wonder then, that most practicing engineers are lost in a fog when it comes to deciding which to choose between the ordinary flow formulas, to say nothing of those that involve the use of Reynolds number.

The writer pleads guilty to having been among the majority (i.e. one of the confused masses). An argument with a capable hydraulic engineer about the use of Kutter's **"n"** in pressure pipe calculations brought about the necessity for digging in and making some studies. It is quite probable that the conclusions reached are somewhat at variance with modern conceptions of flow and hence will probably be viewed with skepticism by the present-day writers on the subject of hydrodynamics.

Genealogy of Pipe Flow Formulas

Two centuries or more ago Chezy produced his well-known formula for flow in pipes in the form

$$V = C\sqrt{rs} \qquad \text{Eq. 1}$$

in which

V = velocity in feet per second.
C = coefficient of roughness as used by Chezy, Williams & Hazen or Mills.
r = hydraulic radius in feet.
S = slope of hydraulic gradient in feet per foot.

Darcy (and Weisbach) transposed the terms in Chezy's formula and put it in the form

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{Eq. 2}$$

in which

h_f = loss of head in feet.
f = coefficient of roughness as used by Darcy & Weisbach.
L = length of pipe in feet.
D = diameter of pipe in feet.
g = gravitational acceleration.

The Author

thus bringing in the proposition that water flow may be likened to that of a falling body (by virtue of the $\frac{V^2}{2g}$ term.)

Many engineers fail to emphasize that these two equations are the same, merely changed by transposition.

For ease in computation most users have adopted the further transposed form

$$h_f = KL \frac{V^2}{D} \qquad \text{Eq. 3}$$

in which

K = coefficient of roughness in exponential type formulae.

or that for each thousand feet of pipe

$$H = K' \frac{V^2}{D} \qquad \text{Eq. 4}$$

in which

H = loss of head, in feet per thousand $= \frac{1000\ h_f}{L}$

K' = 1,000 K.

Once this transposition was done, experiments made in the latter half of the 19th Century and shortly thereafter led many hydraulicians to find all sorts of results indicating that the formula might be written

$$h_f = KL \frac{V^z}{D^x} \qquad \text{Eq. 5}$$

or for 1,000' of pipe

$$H = K' \frac{V^z}{D^x} \qquad \text{Eq. 6}$$

in which

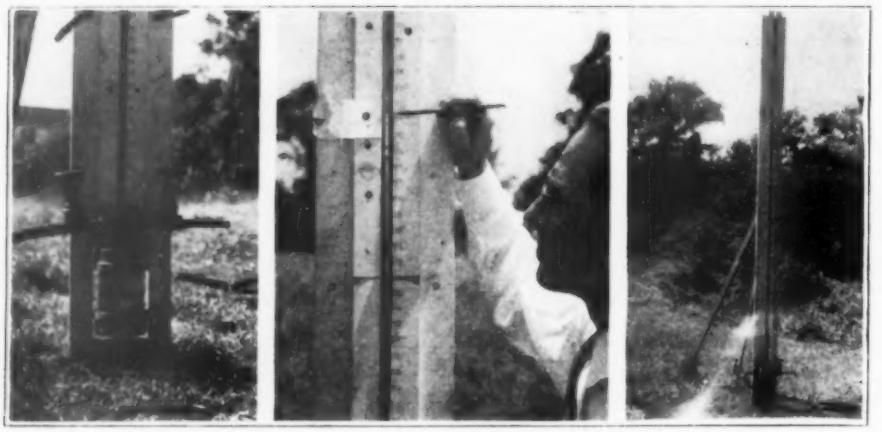
z = Exponent of V in exponential equation
x = Exponent of D in exponential equation

where z varied anywhere from 1.75 to 2 and x varied from 1.0 to 1.33. The practice of using arbitrary values for these two exponents became common and the student was confronted with a maze of formulas from which he might be asked to pick one and substantiate his choice. For practical purposes it may be said that nearly every exponential formula is one of the family group represented by equation (5).

So it is with the Williams and Hazen formula which, when expressed in the form of equation (6), gives a value to z of 1.852 and a value to x of 1.167. This formula received its great acceptance for three reasons (a) the authors of it combined all sorts of tests to arrive at a mean value; (b) the special slide rule created for it greatly simplified use of the formula; (c) the book of tables published gave every one a ready tool for calculation of any flow problem. It is a good average formula and has served a very useful purpose, but it is no better than average and is less accurate for many purposes, particularly for large pipes, than some other formulas.

Viscosity and Turbulence

To Poiseuille must go the credit for determining the effect of viscosity and temperature on fluids in motion. Later, (1883), Reynolds [a] published his justly famous treatise on laminar and turbulent flow in which he showed that each type occurred under certain specific conditions, although the transition zone covers a



Mercury Column Apparatus for Measuring Height of Hydraulic Gradient

rather wide range. His formula, developed as a result of experiments with glass tubes varying from one to three inches in diameter, was

$$R = \frac{LV\rho}{\mu} \qquad \text{Eq. 7}$$

in which

R = Reynolds number = $\frac{\text{Velocity x length}}{\text{Kinematic viscosity}}$

ρ(rho) = Density of fluid.

μ(mu) = ratio of intensity of shear τ (tau) to rate of deformation. (Rouse)

but is more commonly written

$$R = \frac{VD}{\nu} \qquad \text{Eq. 8}$$

in which

ν(nu) = Kinematic viscosity.

Actually Reynolds injected dye from a very small-bore outlet in the center of each tube tested and let this dye demonstrate the nature of the flow, as shown in Fig. 1. He found that the dye remained in linear positions until velocities became sufficient to disturb these positions. Relationships between these various factors changed with diameter of pipe and Reynolds found that when the number R was less than 2,000 all flow was laminar, while above 20,000 all flow was turbulent.

It may be well at this juncture, to emphasize the fact that in practical water works operation we may assume an 8-inch pipe and a velocity of four feet per second as the lower limits of size and velocity that need concern the average problem. For this combination the Reynolds number will be 216,800 at 15° C. This is so clearly in the turbulent zone that there can be no question regarding that phase of the matter.

It therefore goes without saying that the laminar and transition zones mean little or nothing in the normal everyday working problems of the water works operator. This is also tantamount to saying that within the limits commonly encountered in water supply transmission and distribution, there is no such thing as purely laminar flow or conversely, all flow in normal pipes is turbulent. Obviously, if Reynolds' theory is correct, a container of infinite size will have turbulent flow regardless of velocity.

Mills' Formula

In 1872, prior to publication of Reynolds' theory, Hiram Mills began his experiments and completed most of them before Reynolds made his results known. His work [2] published in 1923, proposed the unique theory that friction loss in general is proportional to the first power of the velocity up to a velocity of approximately *2* feet per second for a pipe ⅛-in. in diameter and thereafter varies as the second power. (Mills expressed this change point as the critical velocity.) His fundamental formula may be expressed in the form

$$H = \frac{CV + C_1V^2}{D^{1.25}}$$

in which

H = loss of head, in feet per thousand

C and C_1 are coefficients of the V and V^2 terms, respectively.

V = velocity in feet per second

D = diameter of pipe in feet.

Gavett [3] has shown that in the five sizes of small pipes for which Mills determined the critical velocity, i.e. from 2.07 ft. per second for a ⅛-in. pipe to 0.1 f.p.s. for a 3-in. pipe, the Reynolds number was near 2,000. This seems a rather unique corroboration of the Reynolds theory and is believed to have antedated the work of Reynolds. On the other hand, Mills obviously recognized the decrease in critical velocity as diameter increases and it is clear that in commercial sizes of distribution mains the critical velocity is almost infinitesimal. Even though he proceeded to calculate his formula in full for all sizes of pipe, Mills showed clearly that for all practical purposes the **V** term in his equation could be merged with the $\mathbf{V^2}$ term in the form

$$H = K\frac{V^2}{D^{1.25}} \qquad \text{Eq. 10}$$

The error involved was negligible as Mills pointed out repeatedly in his text.

Recent Formulas

The work of Nikuradse [4] on tests of small pipes artificially coated with sand grains of measured size opened up a new vista of the entire field of hydromechanics. Most of the formulas emanating from those experiments assume that the Chezy-Darcy equation is fundamental and proceed to use the Darcy **"f"** as though it were indisputably correct.

Rouse [5a] (P. 205) gives the vonKarman-Prandtl resistance equation from the Nikuradse experiments for turbulent flow in rough pipes as

$$\frac{1}{\sqrt{f}} = 2\text{Log.}\frac{r_o}{e} + 1.74 \qquad \text{Eq. 11}$$

in which

r_o = Radius of Pipe

e = The roughness height parameter, i.e. height of surface roughness or some linear dimension representing height, form and distribution of such projections.

For smooth pipe, turbulent flow Hinds [6] gives the vonKarman, Nikuradse equation as

$$\frac{1}{\sqrt{f}} = 2\text{ Log}\frac{R\sqrt{f}}{2.51} \qquad \text{Eq. 12}$$

For rough pipes (complete turbulence) the same source shows the Nikuradse, Prandtl, vonKarman equation

$$\frac{1}{\sqrt{f}} = 2\text{ Log.}\frac{3.7}{e/D} \qquad \text{Eq. 13}$$

and the Colebrook equation for rough pipes

$$\frac{1}{\sqrt{f}} = -2\text{ Log}\left(\frac{e/D}{3.7} + \frac{2.51}{R\sqrt{f}}\right) \qquad \text{Eq. 14}$$

It is obvious that equation 14 is merely a combination of equations 12 and 13. Hinds further explains that for completely turbulent flow the Moody [7] limitation is

$$\frac{1}{\sqrt{f}} = \frac{R}{200}\left(\frac{e}{D}\right)$$

and that the term containing R in equation 14 may be ignored when R exceeds that of equation 15. Hinds is one of the few recent authors who has drawn attention to the fact that while there is considerable basic support for the Colebrook equation, other simpler methods may give practical results within a very small per cent of error.

This being so, it may be well to inquire as to some of the reasons therefor as well

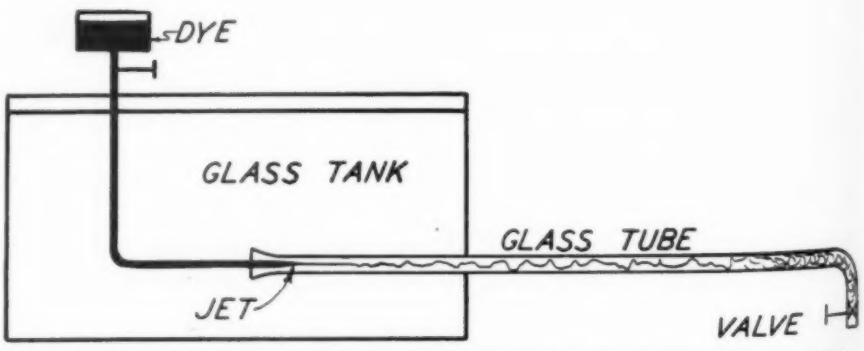


Fig. 1. Reynolds' Apparatus for Demonstrating the Instability of Flow Through a Tube

as to question the utility of such a cumbersome mathematical relation.

Reynolds Theory

In making his experiments, Reynolds used tubes of small diameters, the maximum size having been three inches. He exercised infinite care in avoiding any extraneous disturbances, frequently allowing a tank to remain quiescent over night in order to avoid such effects, particularly those that might be caused by inertia or temperature. Reynolds found that the point at which turbulence was noted traveled up the tube toward the point of entrance (but never reached the entrance) as velocity increased.

A similar phenomenon was noted by Rouse, who carried the theory further to the extent of showing the development of the boundary layer along a flat plate. Rouse [5a] (P. 183) says, "Indeed, although viscous resistance to deformation must, mathematically speaking, extend throughout the moving fluid in all cases, it so happens that at high values of **R** such resistance will be appreciable only in the vicinity of the boundary." Rouse gives Prandtl credit for this.

In reducing some of the theory of hydromechanics to language that may be more easily read, it seems sufficient to say that while the various curves available show conditions of laminar flow, transition zone and turbulent flow, actually all of these three zones exist in normal water works pipes. The only cases where truly laminar flow can occur, are those in which the Reynolds number is small either because of small diameter or low velocity or both. (This presupposes that the fluid is water of fairly constant temperature where viscosity is nearly constant.)

There is ample evidence to support the belief that flow in very small bore tubes is all laminar. This is caused directly by the contact with the sides and might be called "contact flow" in place of "laminar flow". This is the area or zone in which the loss of head is proportional to the first power of the velocity. With a large pipe, the turbulent flow in the major portion of the conduit demonstrates a loss of head proportional to the second power of the velocity and this area might be called that of "free flow". Between the areas of "contact flow" and "free flow" is the transition zone. It is the belief of the writer that the relationship of loss of head to velocity in the transition zone passes through an infinite number of relationships between the first and second powers of the velocity. This would readily account for the fact that the Saph and Schoder experiments, which were made with small bore pipes, substantiated an average exponent for **V** of 1.75. The only point that cannot be explained is that raised by Scobey, who frequently found a loss in large pipes that would indicate an exponent for **V** of more than 2.0. This is apparently not the exception that proves the rule. In fact the only logical explanation that can be suggested is the possibility that the limitation of accuracy of observations precludes too precise a determination of losses within a range of two to five per cent.

Use of Darcy or Weisbach Formula

The use of Reynolds number emanated from Europe where the Darcy formula has been employed for so long that it is frequently accepted as being correct per se. It has the fundamental concept of a falling body because of the $\frac{V^2}{2g}$ term. It also assumes that the loss of head is inversely proportional to the diameter. The first of these two concepts appears to be entirely logical. The second is not generally borne out by practical experience. Nevertheless most of the curves of the Stanton diagram type, as explained by Miles [3], use a logarithmic plotting of Darcy's "**f**" and Reynolds "**R**", presumably on the assumption that the Darcy formula is correct.

All of the divergences found between various recent formulas boil down to the fact that those of the Nikuradse, von-Karman and Colebrook type are merely methods to establish a value of "**f**" for the Darcy formula and that the exponents for "**V**" and "**D**" in the latter are 2 and 1 respectively. In the so-called exponential formulas, the exponents of "**V**" vary from 1.75 to 2 while those of "**D**" vary from 1 to 1.33. The outstanding and unique exception is that of Mills wherein **V** and $\mathbf{V}^2$ terms are both incorporated.

Hinds [6] points out that while the older formulas are imperfect and limited in scope, their uncertainties "are not markedly greater than other uncertainties inherent in the design of water supply systems."

Applicability of Theory

Suppose that the boundary layer of laminar flow at a given velocity, and with given side walls, is 0.01 inch in depth. Then in a pipe 0.02 inch in diameter, all flow under these conditions would be laminar, i.e. one hundred per cent of the cross-section area would be included. For the same conditions of velocity and side walls, the laminar flow in a 10-inch diameter pipe would occupy an annular space equivalent to only 0.19 per cent of the cross-section area. The ratio obviously decreases greatly in proportion as the diameter increases.

Rouse [5b] has clearly demonstrated that the velocity curve, starting at zero at the side walls, varies in a logarithmic relation to the distance away from the wall. This is the fundamental reason for the wide variation in the exponents of **V** found by numerous hydraulicians with various sized pipes. The classic experiments of Saph and Schoder [1] were made with small bore pipes and it is interesting to note that they derived an exponent of 1.75 for **V**—the smallest found in the more commonly known formulas. It seems probable that the high percentage of laminar flow in those experiments was directly responsible for such results. In connection with this, it may be noted that while the theoretical boundary layer is very thin, the effective layer where "**z**" is less than 2 may be appreciable.

The Williams and Hazen formula appears to have been the most comprehensive effort, up to the time of its derivation, to cover a wide range of diameters. These varied principally from 1-inch to 60-inches in diameter although tests of smaller and larger pipes were both considered and the authors pointed out that separate formulas were more applicable to such cases.

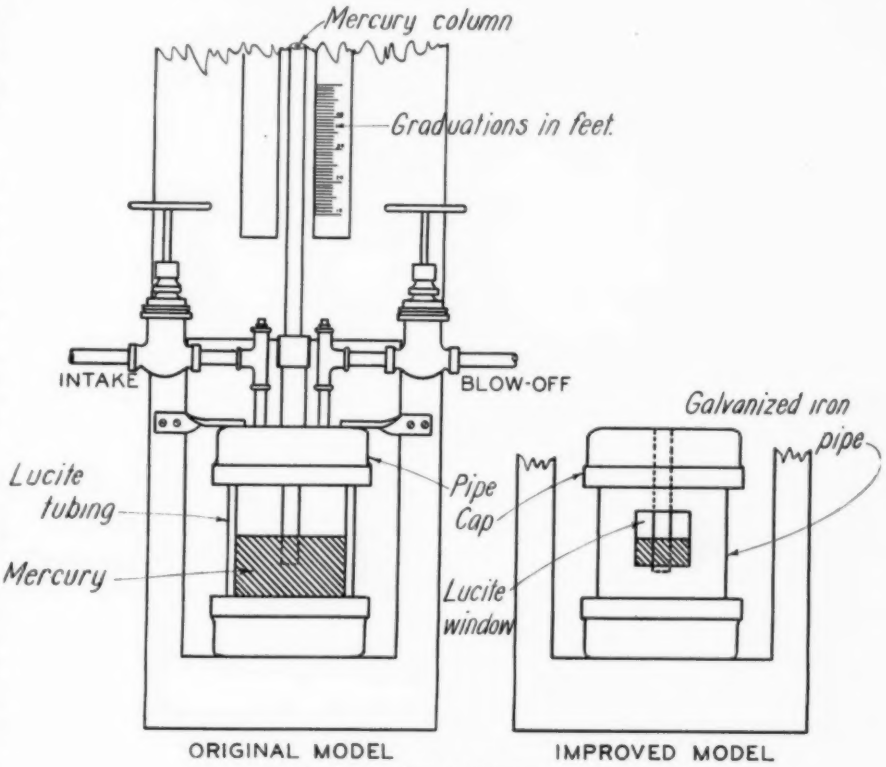


Fig. 2. Design of Mercury Pot

The preponderance of evidence, and the teachings of almost all of the present day writers on hydromechanics, is to the effect that the $\frac{V^2}{2g}$ term may be firmly embedded in the theoretical determination of fluid flow.

The writer in the past has leaned toward the use of the Mills exponent of 1.25 for **D** instead of unity as in the Darcy formula. Careful consideration of this over a period of years leads to the belief that no one exponent can accurately express a true overall relationship. The previously mentioned effects of the laminar layer, as well as the logarithmic velocity curve, both coincident with and adjacent to this layer, lend strong support to the belief that all one may hope to do is to arrive at a reasonable mean for most conditions.

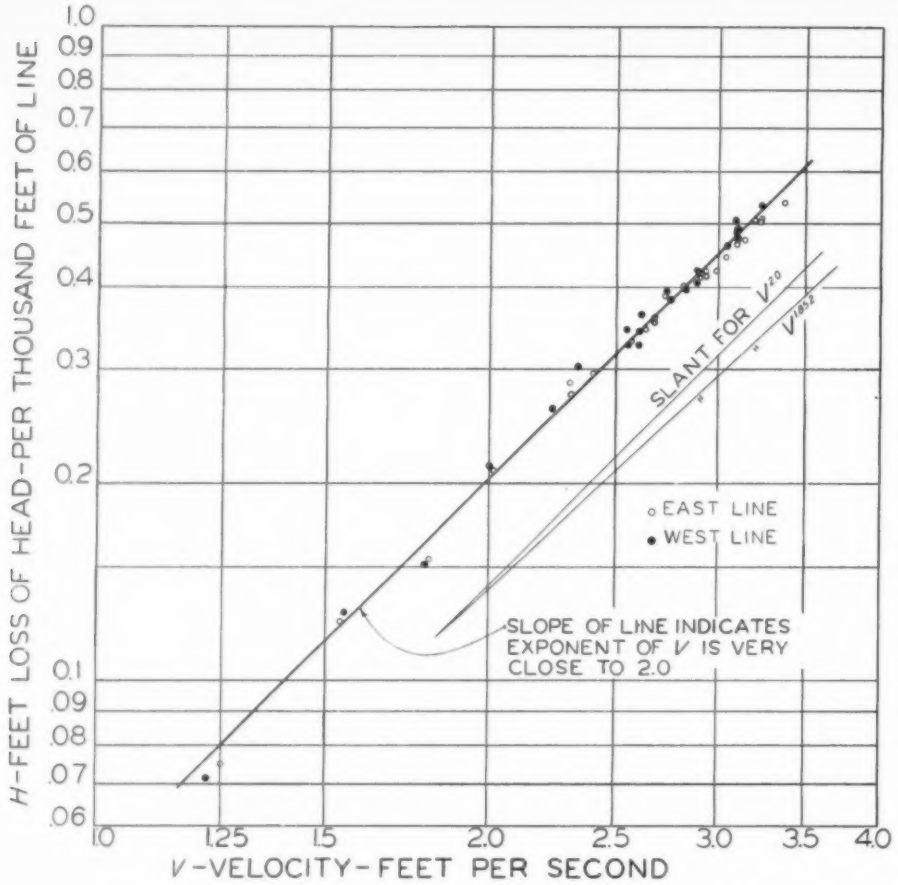


Fig. 3. Relationship of Loss of Head to Velocity in Wanaque Line

If the Miles[9] adaptation of the Stanton diagram is accepted, two difficulties are immediately encountered. The first is the necessity of "guessing" (scientifically, perhaps) at the type of pipe under consideration. After that is done the Darcy formula must be accepted as being correct. When the wide variation in range of diameters normally used by water works men is considered, it seems difficult to believe that the laminar layer, and its attendant velocity influence, can be considered of such uniform effect that the exponent of **D** can be unity throughout a wide range. It appears more likely that the exponent of **D** actually should be a variable. Scobey[10], has suggested that the sum of the exponents of **V** and **D** should equal 3 and he advocated 1.9 and 1.1 respectively for most cases. Thus in the case of the Saph and Schoder experiments, if the **V** exponent is 1.75, the **D** exponent would be 1.25. Schoder[14] later recommended a constant exponent for **D** of 1.25 and a varying exponent for **V**, starting with 1.75 for smooth pipes and increasing to 2.0 for rough pipes.

The work of Hinds[8] and Wyckoff[11] might well be correlated with the other data they are gathering to the end that suitable criteria may be established for large pipe diameters. Possibly this will vary from that of pipes of normal distribution line sizes of say 8-inch to 24-inch diameter. For the time being, the writer believes that the modified Mills formula is as good as any.

It may be seen that with a diameter of unity (i.e. one foot in the English system of measurements) the Darcy formula would be precisely reached, regardless of the exponent of **D**, provided that the exponent of **V** is 2. Assuming that the Schoder and Mills exponent of **D** is correct (i.e. 1.25) the friction loss will be high for pipes under one foot in diameter but will be progressively lower for pipes over one foot in diameter. It is conceivable that as the diameter approaches infinite proportions, the boundary layer has little effect, the only item of importance being that of velocity and the corresponding loss because of turbulence. This idea suggests further that the exponent of **D** might well be a variable, having some such general logarithmic variation as the velocity curve adjacent to the side walls of the pipe. Such an arrangement would complicate the calculation of pipe flows more than the simple exponential form of equation but would not be as difficult to handle as the Colebrook formula—and would probably be more accurate than either.

Practical Results of Pipe Flow Tests

Wyckoff[11] has stated that the California Section of the American Water Works Association is gathering data on tests of many types of pipes. He further explained that unpredictable variations in pressure occur in many, perhaps all, long pipe lines, particularly large diameter pipes. These are thought to have some relation to reflected pressure waves caused by water hammer, perhaps mild in form, but nevertheless noticeable. Wyckoff has suggested that only carefully synchronized time recording devices would establish this fact.

During 1944-46 the writer has been supervising a number of pressure observations and pitot runs on the twin 74-inch Wanaque pipe lines, installed in 1929 and started in operation in March, 1930. Details of these lines were fully described in the *A. W. W. A. Journal* in January, 1941[12].

A photograph of the apparatus used in the pressure tests is shown as well as a sketch, Fig. 2, indicating the principal features of the equipment. The basic design was taken from apparatus devised by Edwin T. Erickson of the Newark Water Department for tests of their Pequannock pipe line, that work having been done under the direction of William G. Banks, Division Engineer. A subsequent perusal of Freeman's book, "Flow of Water in Pipes and Pipe Fittings"[13] shows a similar design on page 44.

Tests of Wanaque Pipe Lines

In the design of the Wanaque pipe lines the usual struggle encountered in estimating friction losses played the same important role that it has in most of such cases. Necessity for observing actual operating conditions as compared with design estimates led to the initiation of a series of tests, a summary of which is given in Table I.

The principal section of mains tested was 58,125 feet long. There are two 74-inch parallel steel, lock-bar mains in this reach, with riveted circumferential joints. Observations were made at each end and at a mid-point which was 30,586 feet from the upper end. The intermediate readings served as a general check on the overall observations. The separate figures for the partial sections are not included and while they substantiate the full reach results, there is unmistakable evidence that the friction loss is slightly greater per 1,000 feet in the portion nearer the reservoir. This is typical of waters of this kind passing through such conduits.

In the entire reach of 58,125 feet there are four cross-connection chambers, three upstream of the mid-point and one below. In each of these chambers the pipe lines reduce to a 48-inch diameter with two 48-inch valves in each line.

Separate observations of the loss in each cross-connection chamber were made, the results being given in Table II. For the purpose of comparing the coefficient of the straight pipe with similar installations it is possible to compute the loss through all of the chambers and to deduct this from the overall loss. This was done in some instances but does not affect the general conclusions and has been omitted here.

The principal stumbling block in all of this work was the constantly fluctuating hydraulic gradient noted also by Wyckoff[11]. In general the maximum deviation at any one point was the equivalent of about 9 inches of water. Of course, the mercury column reduced this proportionately, but it placed a premium on the personal equation in reading the results. Synchronizing of watches was of no avail in attempting to overcome the differences and the best that could be done was to take a mean of the column levels at the time of reading. At the A.W.W.A. conference in San Francisco, Scobey showed a method for snubbing out these pulsations.

Observations were made every five minutes for one hour at each location on the east pipe and then connections were shifted to the west pipe for a similar period. Simultaneous readings were taken on the Venturi meter that registered the combined flow in the two lines. Careful checks showed that the relationship of flow in the two lines was not always constant. Pitot readings did not solve all of these difficulties but permitted an approxi-

mate curve to be drawn showing the varying relationship of the per cent of flow carried in each pipe to the total flow. From this curve the appropriate factor was applied in each instance and results were modified accordingly.

Fig. 3 shows the relationship of loss of head to velocity. It is unfortunate that more points are not available in the lower range but the ones shown were obtained only on week-ends when some of the users were willing to stop withdrawals for a sufficient period to permit observations. Nevertheless there is an unmistakable trend toward the V^2 relationship. This is presented as evidence for one large pipe line which substantiates the theory expressed previously herein. Camp[18] has stated that "In turbulent flow, the velocity gradients, and hence the friction loss, vary almost as the square of the mean velocity in the pipe."

These tests do not establish the value of the exponent of **D**. As previously stated such an exponent is believed to be a variable which, while possibly susceptible of mathematical derivation, might better be first obtained from a complete summation of existing data, together with further experiments to be made in ranges that are not covered by information available at the present time. It is important that such results should be carefully considered from the viewpoint of similarity of factors such as pipe lining, temperature, bends and valves or other disturbing appurtenances.

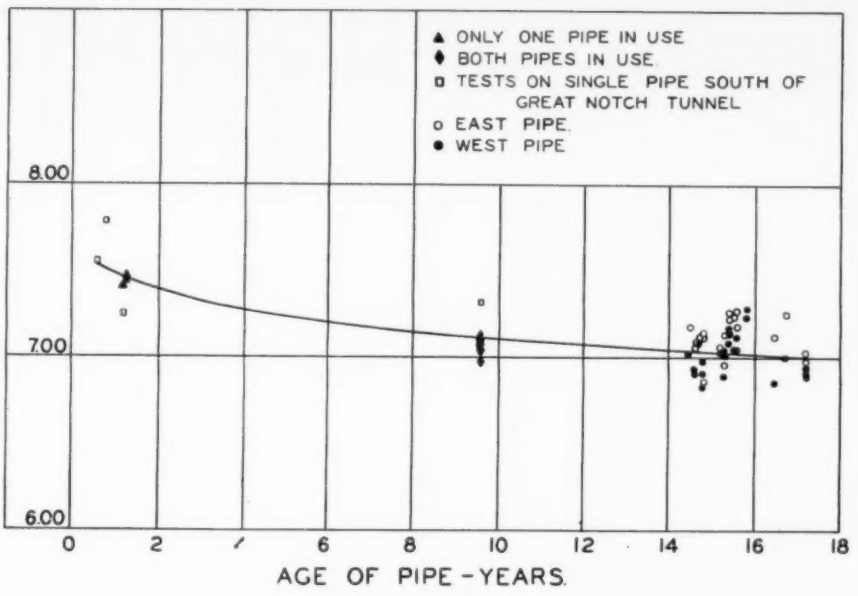


Fig. 4. Loss of Carrying Capacity of Wanaque Lines

Loss with Age

Perhaps no single item in connection with pipe flow has embarrassed engineers more than the change in friction loss with age. Mature study and voluminous writings have both added to the fundamental knowledge and yet it still remains as the one item on which few engineers are reasonably agreed.

Fig. 4 is exhibited to show the results on the Wanaque pipe lines. It is not necessarily to be considered as an ultimate criterion of this type of pipe line. The variations indicate clearly what is to be expected. No results have been omitted and no apologies are offered. As taken from this curve the loss in carrying capacity to date is around eight per cent in 17 years. This is less than has customarily been used in pipe line design.

Recent Data

The recent comprehensive work of Hickox, Peterka and Elder[36] deserves recognition and offers a rare opportunity to observe results on a large scale at high velocities. The somewhat high variation in **f** with respect to **V** as encountered in the Wanaque results led to the plotting of Fig. 5 showing a comparison of these various recordings. It becomes rather apparent that proportionately large variations in **f** are to be expected at the lower velocities particularly those encountered in normal water works practice. This being the case it is small wonder that such an infinite variety of results have been obtained by experiment.

No comments on topics of this nature would be complete without mentioning the work of Bakhmeteff[17] and of Prandtl and Tietjens[18] as well as the various contributions by such hydraulicians as Blasius, Stanton and von Karman.

All of this was brought up to date by Hinds[6] who points out further that the exponential type of formula produces a straight line on the Stanton type diagram. There is, however, a peculiarity of the Colebrook formula which should be mentioned. This is that the value of **R** approaches infinity at a certain value of **f** and then becomes minus as the value of **f** diminishes. No particular significance need be attached to such a case, as it is unlikely that any commercial pipe can be produced that would be capable of attaining a smoothness of this order.

Much has been done by Covil[19] to explain the questions presented herein and his efforts to simplify pipe flow computa-

TABLE I.

Flow Test of Wanaque Aqueduct.

Date	Pipe	Loss in Head per 1000 ft.	Q mgd.	V Ft./Sec.	f Darcy Formula	C w.&h. Williams and Hazen Formula	C Fm. Hills Formula	C Colebrook Formula	C Omitting R Term of Colebrook	R (Reynolds)	T Temp of Water in °C.
1944											
7/25	E.	.467	59.63	3.094	.0194	112	1.453	.00559	.00587	1,435,000	12°
	W.	.428	55.72	2.891	.0203	110	1.418	.00677	.00707	1,341,000	"
8/16	E.	.402	54.25	2.815	.0201	111	1.425	.00652	.00679	1,447,000	16°
	W.	.394	52.75	2.737	.0209	109	1.399	.00767	.00795	1,407,000	"
8/30	E.	.360	51.59	2.677	.0199	111	1.432	.00625	.00653	1,487,000	19°
	W.	.363	50.45	2.618	.0210	109	1.396	.00781	.00808	1,455,000	"
9/18	E.	.346	50.91	2.642	.0197	112	1.441	.00599	.00627	1,468,000	19°
	W.	.342	50.31	2.611	.0199	111	1.432	.00625	.00653	1,451,000	"
10/4	E.	.330	49.59	2.573	.0198	111	1.438	.00608	.00638	1,356,000	17°
	W.	.327	48.34	2.508	.0206	110	1.407	.00718	.00748	1,322,000	"
10/25	E.	.296	46.44	2.410	.0202	111	1.421	.00656	.00691	1,143,000	13°
	W.	.302	45.98	2.386	.0210	110	1,392	.00773	.00808	1,132,000	"
11/8	E.	.286	44.52	2.310	.0213	110	1.385	.00815	.00854	1,010,000	10°
	W.	.285	44.23	2.295	.0215	109	1.379	.00847	.00886	1,004,000	"
1945											
3/14	E.	.423	55.80	2.896	.0200	111	1.429	.00624	.00664	1,009,000	2.4°
	W.	.386	53.09	2.755	.0202	111	1.423	.00649	.00691	960,000	"
5/2	E.	.389	52.65	2.732	.0207	109	1.405	.00733	.00765	1,239,000	11.4°
	W.	.345	49.11	2.548	.0211	109	1.392	.00792	.00826	1,155,000	"
5/31	E.	.425	56.56	2.935	.0196	111	1.444	.00582	.00612	1,351,000	11.8°
	W.	.384	52.96	2.748	.0202	110	1.416	.00659	.00691	1,265,000	"
6/7	E.	.356	51.69	2.682	.0196	111	1.441	.00579	.00612	1,244,000	12°
	W.	.326	49.23	2.555	.0198	111	1.435	.00603	.00638	1,185,000	"
6/20	E.	.447	58.52	3.037	.0192	112	1.457	.00535	.00563	1,441,000	13°
	W.	.463	58.54	3.038	.0199	111	1.433	.00625	.00653	1,441,000	"
6/25	E.	.472	60.43	3.136	.0190	114	1.465	.00509	.00537	1,511,000	13.5°
	W.	.474	59.74	3.100	.0196	112	1.446	.00585	.00612	1,494,000	"
7/11	E.	.418	56.68	2.941	.0192	114	1.460	.00534	.00563	1,417,000	13.5°
	W.	.408	55.53	2.882	.0195	113	1.447	.00568	.00598	1,389,000	"
7/30	E.	.503	62.26	3.231	.0191	112	1.462	.00524	.00550	1,620,000	15°
	W.	.485	59.55	3.090	.0202	110	1.425	.00665	.00691	1,549,000	"
8/27	E.	.415	56.15	2.914	.0194	113	1.452	.00561	.00587	1,536,000	17°
	W.	.398	54.51	2.829	.0197	112	1.438	.00599	.00627	1,491,000	"
8/31	E.	.539	64.95	3.370	.0189	109	1.473	.00504	.00527	1,776,000	17°
	W.	.534	62.39	3.238	.0202	106	1.421	.00668	.00691	1,707,000	"
11/23	E.	.0750	24.08	1.250	.0190	122	1.464	.00461	.00537	547,000	10°
"	W.	.0715	23.59	1.224	.0191	122	1.466	.00472	.00550	535,000	"
	E.	.154	34.75	1.803	.0188	119	1.475	.00461	.00514	789,000	10°
	W.	.152	34.50	1.790	.0188	120	1.472	.00461	.00514	783,000	"
1946											
8/5	E.	.508	61.71	3.202	.0197	111	1.441	.00601	.00627	1,619,000	15.3°
	W.	.507	59.29	3.077	.0213	107	1.386	.00828	.00854	1,555,000	"
10/8	E.	.425	57.46	2.982	.0190	114	1.467	.00512	.00537	1,657,000	19°
	W.	.426	55.54	2.882	.0204	110	1.416	.00694	.00719	1,601,000	"
1947											
4/5	E.	.123	29.63	1.538	.0206	114	1.406	.00677	.00748	565,000	4.0°
	W.	.127	29.82	1.547	.0211	114	1.394	.00757	.00826	568,000	"
4/5	E.	.209	38.82	2.014	.0204	112	1.414	.00665	.00719	739,000	4.0°
	W.	.212	38.57	2.001	.0210	111	1.395	.00755	.00808	735,000	"
4/5	E.	.274	44.55	2.312	.0203	111	1.418	.00660	.00707	849,000	4.0°
	W.	.262	43.06	2.234	.0208	110	1.400	.00733	.00778	880,000	"
6/30	E.	.508	62.42	3.239	.0192	112	1.458	.00536	.00563	1,502,000	12.0°
	W.	.490	59.65	3.095	.0203	110	1.418	.00679	.00707	1,435,000	"
10/7	E.	.422	56.31	2.922	.0196	112	1.442	.00586	.00612	1,540,000	17.0°
	W.	.423	55.29	2.869	.0204	110	1.416	.00693	.00719	1,512,000	"

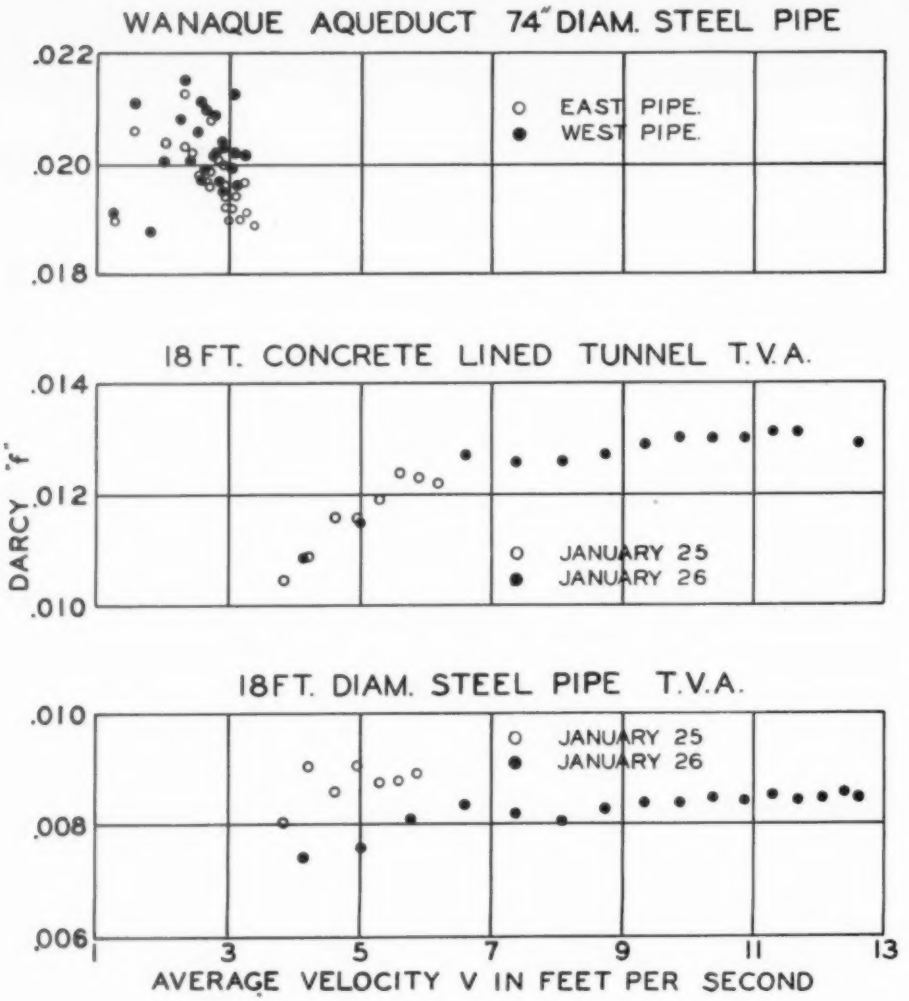


Fig. 5. Comparison of Wanaque Results with Other Pipes

tions must be accorded considerable weight. It is important to note that Covil expressed the opinion that good information is lacking regarding large diameter pipes and that the exponent of **V** may increase with the size of pipe but is constant for one diameter. He further suggested values between 1.88 and 2.03 for large diameters.

Summary and Conclusions

1. Practically all normal water works flow problems involve turbulent flow.

2. With the tendency of hydraulicians to use Reynolds number in pipe flow calculations, it becomes highly important to determine the accuracy of the Darcy (or Chezy) formula inasmuch as this is the commonly accepted basic equation used in such cases.

3. According to the Moody limitation, the use of Reynolds number, in the case of the large pipes tested, may be ignored in the Colebrook equation. This is substantiated by the results shown in Table I.

4. Flow in the vicinity of the pipe walls usually termed "laminar flow" might also be called "contact flow" while near the center the flow usually termed "turbulent flow" may be called "free flow." Between the two is a transition zone with "mixed flow."

5. The plotting shown in Fig. 3 (taken from results in Table I) gives substantial backing to the belief that the exponent of **V** in the exponential type of flow equation should be 2 for large diameter pipes instead of the lesser values more commonly used. A careful consideration of the facts and theories presented here gives substantiation to the probability that while exponential equations of various types are generally accepted, many of them are not as accurate as the Darcy formula for large pipes.

6. It is believed that for any one large pipe, the Darcy formula, once the coefficient has been determined, is more accurate than many other formulas.

7. It seems probable that to be truly accurate an exponential formula should read somewhat as follows:

$$H = \frac{KV + K_1V^{1+a} + K_2V^{1+a_1} \ldots\ldots\ldots\ldots + K_nV^2}{iD^{1.25} + i_1D^{1.25-b} + i_2D^{1.25-b_1} \ldots + i_mD^{1.25-b_m}}$$

If this expression be true, then a ready explanation is available for the many exponential formulas offered by various hydraulicians in the past. The writer is convinced that for practical purposes, in the case of large diameter pipes, all but the K_nV^2 term can be eliminated in the numerator of the right hand side. It appears that additional experimental data are needed to establish the best average value for a single term to be used in the denominator.

8. For most practical purposes, it is believed that flow in large diameter pipes can best be expressed by the simple formula

$$H = K^1\frac{V^2}{D^x}$$

(where x may be between 1 and 1.25) or by the original Chezy or Darcy relation

$$H = K^1\frac{V^2}{D}$$

Symbols

- **a** Part of the exponent of **V** in theoretical exponential equation
- **b** Part of the exponent of **D** in theoretical exponential equation
- **C** Coefficient such as Chezy, Williams & Hazen or Mills
- **D** Diameter of pipe
- **e** Some authors use **k** or ϵ (eta) for this symbol). Height of surface roughness or some linear dimension representing height, form and distribution of such projections.
- **f** Darcy-Weisbach resistance coefficient
- **g** Acceleration of gravity
- **h** Loss of head due to friction
- **K** Resistance coefficient in exponential type formula
- **K¹** Resistance of 1,000 feet of pipe = 1,000 K
- **L** Length of pipe
- **R** Reynolds number = $\frac{\text{Velocity x length}}{\text{Kinematic viscosity}}$
- **S** Slope of water surface
- **H** Loss of head, in feet per thousand = $\frac{1{,}000\ hf}{L}$
- **i** Coefficient of **D** in theoretical exponential formula
- **r** Hydraulic radius
- **r_o** Radius of pipe
- **z** Exponent of **V** in exponential equation
- **x** Exponent of **D** in exponential equation
- **V** Mean velocity
- **ν** (nu) Kinematic viscosity
- **ρ** (rho) Density
- **μ** (mu) Ratio of intensity of shear to rate of deformation or viscosity coefficient

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TABLE II

Loss Through Connection Chambers

Line	Station at Center Line of Cross-Conn. Chamber	Total Length Between Hose Connections to Aqueduct Feet	Total Loss of Head in feet	Length of Approaches Feet	Loss of Head in Approaches Feet	Net Loss of head Thru Connection Chamber Feet	Velocity in Pipe Feet per Sec.	No. of Velocity Heads Lost thru Connection Chamber
	83+96	612						
East			0.705	558	0.221	0.484	2.91	3.70
West			.713	558	.221	.492	2.94	3.67
	296+81	580						
East			.609	526	0.204	.405	2.91	3.10
West			.587	526	.204	.383	2.91	2.92
	391+75	300						
East			.412	246	0.094	.318	2.84	2.54
West			.472	246	.094	.378	2.84	3.03
	423+10	230						
East			.442	176	0.067	.375	2.80	3.07
West			.371	176	.067	.304	2.84	2.43

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S. B. Morris Appointed To Colorado River Board

Samuel B. Morris, General Manager and Chief Engineer of the Department of Water and Power, Los Angeles, Calif., has been appointed to the Colorado River Board of California by Governor Earl Warren. This board will study the use and availability of the Colorado River. Mr. Morris presented a paper on the history and significance of the Colorado River at the San Francisco meeting of the A.W.W.A. in July, 1947.

Regular Corps Appointments for Engineer Officers in U.S.P.H.S.

Competitive examinations for appointments in the Regular Corps in the U. S. Public Health Service in grades of Assistant Sanitary Engineer (1st Lieutenant) and Senior Assistant Sanitary Engineer (Captain) will be held during June, 1948. Approximately 15 appointments are to be made.

Entrance pay for the assistant grade with dependents is $3811 a year and for the Senior Assistant Grade with dependents is $4351 a year. Promotions are at regular intervals up to and including the grade of Senior Sanitary Engineer, which corresponds to the rank of Lieutenant Colonel, at $7018 a year. Promotion to grades above Senior Sanitary Engineer is by selection. Retirement pay after 30 years' service or at the age of 64 is $4950 a year.

Applicants must (1) be citizens of the United States at least 21 years of age; (2) have a degree in one of the several branches of engineering, from a school of recognized standing. Assistant Senior Engineer must have at least seven years educational (exclusive of high school) and professional training and experience; Senior Assistants must have eleven years training and experience.

For further information, write the Surgeon General, U.S. Public Health Service, Washington 25, D.C. Application should be submitted without delay and must be received prior to June 1, 1948. The written examination is scheduled for June 21, 22, and 23, and will be held at designated service stations convenient to the candidates.

Industrial Waste Treatment Course

A course on "Industrial Wastes Disposal and Stream Pollution Control" is being given at the University of Michigan Extension Service, Detroit, Mich.

Classes were started on February 20 and are held for a two and a half hour period each Friday night for a period of 12 weeks. The enrollment fee is $7.50. This unique course is sponsored by the University of Michigan and the subject matter has been prepared under the direction of Professor E. M. Boyce, Head of the Civil Engineering Dept., University of Michigan, and Dr. G. M. Ridenour, School of Public Health, University of Michigan. The course leader is Mr. J. E. Cooper, sanitary engineer of the Ford Motor Co.

Cooperating in this school are the Committees on Education of the Michigan Section of the A.W.W.A., and the Michigan Sewage Works Association. Assisting Mr. Cooper is Austin M. Habermehl, Supervising Chemist of the Detroit Sewage Treatment Works.

This series of lectures which offers an opportunity to many persons in the Detroit area will include lectures on the magnitude and character of the industrial waste problem, waste disposal capacities of streams, governmental abatement activities, how to organize an industrial waste survey, standard treatment methods and equipment, and lectures on specific industrial wastes including gas and coke plant, metal industries, pulp and paper mill, meat packing, canning factory, etc.

Among the lecturers scheduled for this school are Milton P. Adams, Executive Secretary, Michigan Stream Control Commission; John M. Hepler, Director of Michigan Dept. of Health; L. F. Oeming, Sanitary Engineer, Michigan Stream Control Commission; Ernest Boyce, Head of the Dept. of Civil Engineering, University of Michigan; C. L. Palmer, Senior Civil Engineer, City of Detroit; J. E. Cooper, Supervisor Water & Wastes Laboratory, Ford Motor Co.; E. F. Eldridge, Research Associate, Michigan State College; T. H. Powers, Waste Project Engineer, Dow Chemical Co.; and P. F. Morgan, Research Engineer of National Council for Stream Improvement.

Virginia Joins Ohio River Valley Compact

The signature of Governor William M. Tuck of Virginia, March 5, on a bill passed by both houses of the state legislature giving him authority to enter into the eight-state Ohio River Compact, completed final approval of this $300,000,000 project, which Virginia has been accused of holding up.

Promotion of a compact, designed to relieve pollution of the Ohio River, began several years ago in Cincinnati, Ohio, under leadership of the late William Foust Wiley, former publisher of the Cincinnati Enquirer and former president of the Cincinnati Chamber of Commerce. The work has been carried on since by Hudson Biery, who succeeded Wiley.

The huge project will benefit 18,000,000 people in about 200,000 square miles of territory. States participating in the agreement are New York, Pennsylvania, West Virginia, Ohio, Kentucky, Indiana, Illinois, and Virginia.

The next procedure will be the appointment by the governor of each of the states involved of three compact commissioners for each state, and three commissioners will be named to represent the Federal government.

A part of the program calls for a $25,000,000 sewage disposal system for Cincinnati and Hamilton County. —*Don Wimmer.*

RADIO SERVICE IN THE WATER SUPPLY FIELD

Is Your Operation Big Enough for Radio Service? If so You Will Get Some Pointers in this Article.

By WM. C. MORSE
Supt. of Water,
SEATTLE, WASH.

WILL the installation and use of radio save money for my department? Will it improve the efficiency of the department? These are questions that every water department manager should ask; and, by analysis, find the answer.

Unless there exist very unusual conditions, the question may be answered in the negative for any department having less than three service vehicles. On the other hand, the answer is in the affirmative for any department having eight or more service vehicles. For the great number of operations that fall between these two classifications, although the installation of radio deserves consideration it also warrants careful study and analysis of the potential gains vs. costs.

Let us examine into the cost of installation, operation and maintenance of an ordinarily well equipped Radio Service of the F. M. type. The reasons for selecting F. M. as against A. M. have been given many times.

The Author.

They are freedom from interference, absence of static and clarity of transmission.

Frequently the installation cost of the main or master fixed station and the overall operation and maintenance of the complete system can be divided with other service departments such as police, fire, light and power, telephone and transportation. It is the possibility of such a division of costs that will permit many of the medium sized cities to utilize radio to real advantage.

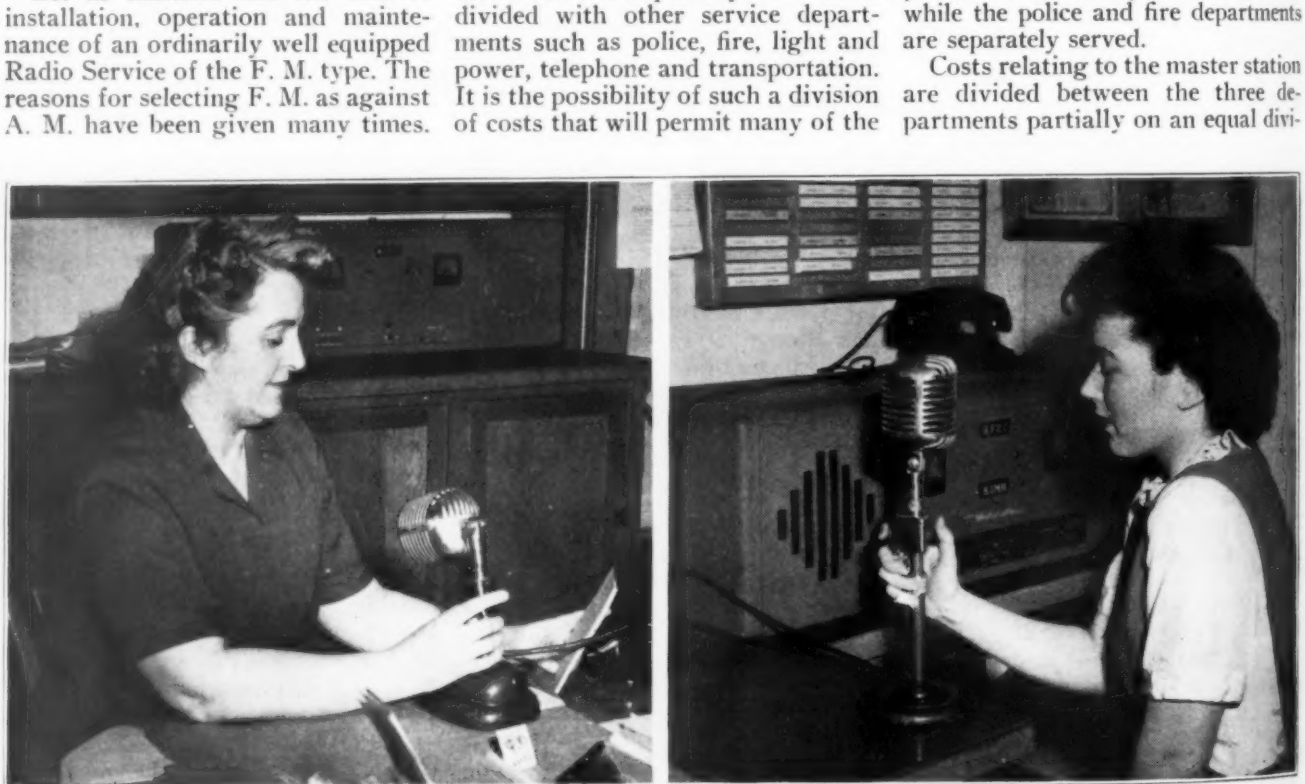
Seattle Costs

I have exact figures of costs covering the city of Seattle, and although I do not have exact figures for other cities, I am led to believe that these costs parallel those of other installations per unit of installation.

The main or master station equipment including antenna and connections but not including a separate building (which ordinarily is not desirable) cost $7,382.46. Auxiliary fixed stations cost $1250.00 each, and each mobile unit (two way) cost $550.00.

In this city the municipally owned water, light and transportation departments use one system in common while the police and fire departments are separately served.

Costs relating to the master station are divided between the three departments partially on an equal divi-



(photos by John Bossert.)

Radio Control Consoles and Transmitters in Two Master Stations.
Messages are sent and received by regular office employees. Light Dept. Station on left; Station in Water Dept. office on right. Telephone is on direct wire to Fire Dept.'s Master Station.

Newest Mobile Field Set.
Operator in communication with office. Receiving speaker behind dash grille; transmitter "mike" hangs on hook to right. Receiving and transmitting units, shown in another picture, are beneath front seat.

sion of the cost and partially through pro-rating total costs, based on the relative number of times each department used the facilities.

Costs relating to the fixed auxiliary stations and mobile units are borne by the particular utility user.

This department's overall cost of operation exclusive of depreciation and obsolescence for 1946 was $1,485.50. Allowing a very high rate for depreciation and obsolescence (16⅔% straight line) our share of the write-off of the cost of the master station has been $246.08 per year. This is equivalent to 20% of 16⅔% of $7,382.46. It will be noted that our share is only one half of that charged to each of the other two utilities associated in the use of the equipment—namely 20% as against 40% charged to the light and transportation departments.

The total cost to the water department for the year 1946, including depreciation on auxiliary and mobile units, was $2,973.54. This covers our portion of the operation of the master station, two fixed auxiliary stations and *nine* mobile units which are driven by supervisory personnel. The total installation of the three departments consists of one master station, seven fixed auxiliary stations and forty-two mobile units. Smaller installations would, of course, cost proportionately more in capital investment and for operation in terms of unit of property value or service.

Operating Personnel

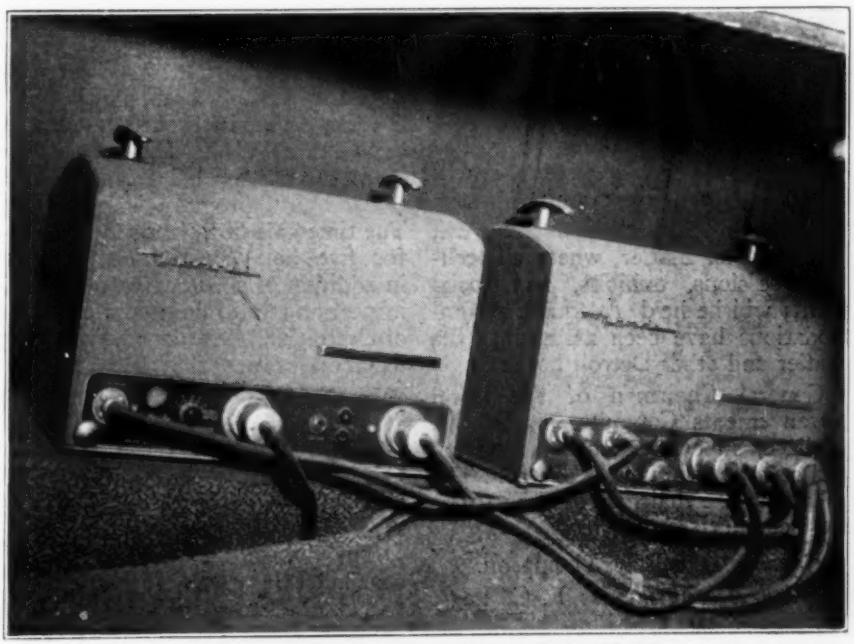
The master station normally is placed where there is continuous attendance. Where this is done no added personnel is required, as one of the regular office employees can readily relay all calls. The location of the fixed auxiliary stations are covered by the same conditions. Should neither the office nor shop be manned twenty-four hours a day, the advantage of Radio Service would be restricted to the open hours.

While in the larger units a man trained in radio mechanics is usually in charge of the equipment and responsible for its good condition, in smaller units almost any mechanically minded man can, with the occasional help of a radio service man, keep the installation in serviceable condition.

With this department to cover the overall Radio Service cost the actual time saved by the use of radio has to average only forty to forty-five minutes per day for members of the supervisory staff, not to mention the saved wear and tear on automotive equipment, gasoline, etc. Aside from the dollar value of the time saved there is the untold advantage of quick coverage of any main leak or break or any other emergency. In the line of public relations a consumer dilemma can be responded to with impressive promptness.

Duty to the Public Angle

Every operator knows how quickly damage may be caused by a broken water main, not only to his installation but to others involved. In some high value areas served by large mains, every minute that can be saved in making a shutoff has an important dollar and cents value in reducing street damage and basement flooding. It is not pleasant to see the water from a broken main pouring into a basement filled with valuable merchandise and realize that damage is accruing by the second. From the standpoint of a duty



Mobile Transmitting and Receiving Units.
Located beneath front seat of Radio Cars, in easy reach of operator.

to the public and public relations it makes little difference whether the cost of this damage finally is borne by the department or not. Someone has to bear it and every possible effort should be made to shorten the time of shutoff.

Here radio has been found to work to great advantage with the time necessary for shutoff reduced in some cases by two thirds. One such break (and every operator is continuously faced with the possibility) if not quickly controlled can cause more damage cost than the cost of radio would amount to in several years.

Courts have generally held that where no negligence can be shown in design, construction or operation of water systems, damages resulting from broken water mains can not be charged against the water department involved, but they have equally generally held that excessive delay in controlling breaks resulting in increased damage to property is negligence, warranting assessment of the cost of damage against the water department.

From a service standpoint every water department is charged with the responsibility of utmost expedition. Can an operator say that he has used utmost expedition if important time is lost in controlling a break if he is not using every time saving device that he can reasonably and justifiably use?

These closing remarks by Mr. Morse serve to reflect those in a recent communication from A.W.W.A. headquarters to members, in which Secretary Jordan emphasizes the following—"Public water supply executives have not been as responsive to the values of radio communication as have been their contemporaries in other utilities. . . . You are requested to consider whether a communication from you to the Federal Communication Commission might not prove of value in holding available for assignment to the utility groups no fewer channels than are now assigned—preferably that the number of channels assigned be increased. . . . Chances for water utilities obtaining assigned bands are now endangered by the ceaseless pressure from commercial interests endeavoring to secure unassigned channels originally reserved for the public service groups." Be assured that letters from water supply executives to Wayne Coy, Chairman, Federal Communications Commission, Washington, D.C., will be given every consideration, whereas continued silence from the water supply industry can only signify disinterest. To delay longer may be too late.—L.H.E.

Quick communication between supervisory operating and maintenance personnel is of great advantage. This is why the telephone has become so necessary in the conduct of business. Radio simply makes instantaneous voice communication available between the supervisory forces, no matter where they may be located, so long as they are in contact with the various mobile units.

If the interested operator will weigh the above given costs, adjusted for the size of the required installation, against the operating costs and projected amounts of damage that may be saved, then he can answer the opening question to a point that will justify him in obtaining exact figures of cost of Radio Service installation and operation. With such information at hand then a final determination will in most cases become obvious.

A Word of Advice

One word of advice. The available bands that can be alloted by the Federal Communications Commission are being rapidly taken up. If any operator is thinking in terms of radio installation his analysis and determination should not be delayed. His application to the Commission in Washington (Wayne Coy, Chairman) for assignment of wave band should be promptly filed in order that those bands reserved for use by the water utility group, and not yet taken up, may not be transferred to other utilities or commercial applicants.

SEWAGE FEDERATION EXPECTS DETROIT CONVENTION TO BE LARGEST YET

Plans for the Twenty-first Annual Meeting of the Federation of Sewage Works Associations, to be held at Detroit, Mich., on October 17-21, 1948, are definitely taking shape. The live-wire Michigan Association will be host to the Federation.

Headquarters for the meeting will be the Hotel Statler, where all technical sessions, exhibits, and social events will be held. Adequate accommodations have been assured at the Statler and other Detroit hotels.

General Chairman of the Local Arrangements Committee is W. M. Wallace, superintendent of filtration and sewage treatment at Detroit. The following are chairmen of the various local committees.

Registration: G. R. Thompson.
Hotel Arrangements: F. G. Legg.
Finance: W. E. Smith.
Inspection Trip: Leroy Smith.
Local Host: W. F. Shephard.
Entertainment: (Chairman to be appointed).

Chairman C. C. Larson of the Publications Committee has appointed F. W. Gilcreas to head a special program committee, and formulation of the technical program is proceeding. The timetable of the meeting provides for five half-day technical sessions in addition to a full afternoon for an inspection trip to the treatment plants of Detroit and Dearborn. One of the technical sessions has been set aside for the new Industrial Wastes Forum, which is expected to be an outstanding feature. As another program feature, the concluding session on Thursday morning, October 21, will be devoted to the Operators' Forum.

The Annual Federation Luncheon scheduled for Tuesday, October 19, will be addressed by a prominent speaker.

General entertainment and social events will include a reception followed by entertainment and a dinner-dance. Another evening function will be the Michigan Dinner.

A large and diversified manufacturers' exhibit is expected to develop under the direction of Arthur T. Clark, secretary-manager of the Water and Sewage Works Mfrs. Assn.

Everyone planning to attend the convention is urged not to delay in making hotel reservations. Rates at the Statler are: single—$3.50 to $5.00 (with shower) and $5.50 to $8.00 (with tub and shower); double—$6.00 to $10.00; double (twin beds)—$7.00 to $10.50; suites—$12.00 to $20.50. Reservations may be placed with the nearby Book-Cadillac Hotel, which has also assigned rooms for the duration of the convention.

Because of the central location of Detroit, a registration exceeding the record of 812 set at Toronto in 1946 is expected.

OPERATION OF PNEUMATIC STORAGE TANKS

by J. B. WILKINSON, M.E.
Chief Engineer
FIRE INS. RATING BUREAU
MILWAUKEE, WIS.

A PNEUMATIC storage tank as used in municipal water works serves the twofold purpose of maintaining domestic pressures within a selected range and of supplying a continuous domestic demand with intermittent operation of a pump.

Low initial cost is amongst the reasons for selecting this type of storage instead of an elevated tank or reservoir. However, pneumatic storage is the most expensive type of water storage in terms of cost per gallon of storage. This is the case because, as will be shown later, the useful capacity within the desired pressure range is usually only a small fraction of the full volume of the tank, and also because the tank must be heavily constructed to withstand the maximum system pressure. It follows that pneumatic storage rarely is large enough to make an important contribution to fire protection, and the usual location is at or near the pumping station. On the other hand pneumatic storage may frequently be justified for betterment to water service on the fringes of water supply systems serving residential areas.

Typical main installations are found in small villages of a few hundred population. The horizontally-mounted tank is cylindrical in cross-section, has dished heads and access manhole, and fabrication conforming to the ASME code for unfired pressure vessels is indicated by stamped marking on the shell. Diameter may be 8 or 9 ft., and gross capacity 16,000 to 20,000 gallons. Larger or smaller tanks are used according to the size of the system. Connection is made from the bottom to a distribution main (a 4 in. pipe for such connection is ordinarily sufficient) and from the top to the compressor used to replenish the cushion air from time to time. The air line should have a renewable-disc angle globe valve at the tank, and toward the compressor a bronze-seat check valve and a brass pop relief valve of the same size as the pipe. The latter should be set to blow at 10 per cent above the maximum operating or design pressure of the tank, whichever is the smaller. A gage glass reveals the air-water level. Automatic operation of a pump is provided for by a pressure-actuated starter, or a float switch can be used.

The Author

Two or more tanks can be operated as a unit by connecting discharge and air lines to common headers. Such installations are not usual because one larger tank will provide equal storage at less cost.

Operating Considerations

Operators of pneumatic storage systems may be uncertain as to where the air-water level should be carried for best results. Operating conditions vary widely and there is no single answer which applies to every case. The objective is to secure the largest possible effective storage with the smallest possible variation from the average pressure wanted in the system. But relatively large volume is associated with a wide pressure range; conversely, close pressure regulation is obtainable only by lessening effective storage.

There is some minimum pressure that must be maintained to provide the desired service. This may be 40 lbs. in a place where domestic supply is the principal consideration, or may be 60 lbs. or more where buildings are high, topography irregular, or sufficient pressure for fire streams is wanted for the short time required to get an operator to a normally unattended pumping station.

Having decided upon the minimum required pressure, the next step is either to select the range of operating pressure, or to determine the storage needed to supply the demand without too frequent starting of the pump. The top pressure will be the governing consideration in some instances—as where certain areas are low and pressures correspondingly high. Or, when the demand is relatively large and approaches the capacity of the pump, the largest possible storage may be wanted to lengthen the operating cycle. But in any given installation, fixing either of these factors automatically affects the other.

Calculation of Pressure-Volume Relations

Air trapped above the water in a pneumatic storage tank is an elastic medium. Pressure-volume relations follow Boyle's law which, briefly stated, is that at constant temperature—approximately true in actual practice—the volume of the confined air multiplied by its absolute pressure gives a number which is always the same for the particular container holding the same amount (weight) of air.

This relation, expressed mathematically, is represented by the formula

$$PV = C,$$

where P is the *absolute** pressure in lbs.

V is the *air volume* in any convenient unit, such as cubic feet or gallons.

C is a *constant*—that is, the fixed number referred to in the preceding paragraph.

To illustrate the application of this formula to a pneumatic equalizer, assume an 18,000-gallon tank containing 9000 gallons of air over water; the pressure gage shows 45 lbs. The volume of the air is wanted when water is pumped in until the pressure is raised to 60 lbs. gage. It will be convenient and sufficiently accurate to assume that atmospheric pressure (which must be added to the gage reading to give absolute pressure) is 15 lbs.

$$\text{C (is the constant)} = P \times V = (45 + 15) \times 9{,}000 = 540{,}000$$

Having determined this constant, the air volume at 60 lbs. gage becomes

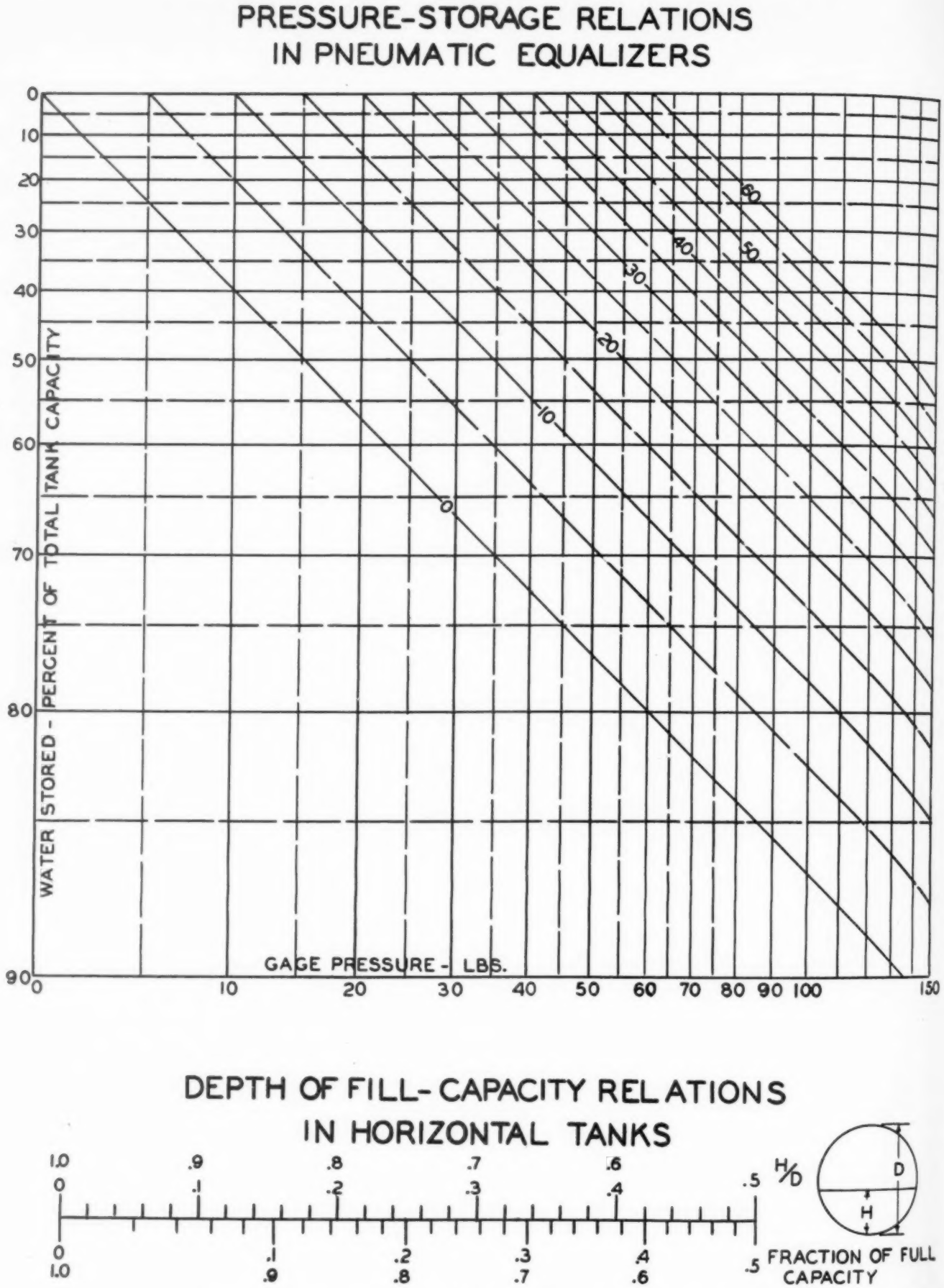
$$V = \frac{C}{P} \frac{540{,}000}{(60 + 15)} = 7{,}200 \text{ gallons}$$

and the water stored is the tank capacity minus the air volume, or 10,800 gallons.

Chart of Pressure-Storage Relations

Numerical calculations of the sort shown are cumbersome, and several must be made to cover the entire operating range. The accompanying Pres-

*Absolute pressure is gage plus atmospheric pressure.



Pressure-Storage and Fill-Capacity Relations in Pneumatic Tanks

sure-Storage Relations Chart has been prepared to make such calculations unnecessary, and at the same time express the relations directly in terms of water volume and gage pressure.

The numbers on the lower horizontal axis are pressures as shown by the gage on the tank. Those on the left margin refer to the volume of water in the tank, expressed as a percentage of the total tank capacity.

If the operating condition at any time is indicated by a point on the chart at the intersection of a water-stored and a pressure line, corresponding pressure-storage relations when water is pumped in or withdrawn will lie along a diagonal line drawn through this point parallel to the several diagonals on the chart. These last are marked 0, 10, etc., the numbers indicating the residual pressure which will remain when the tank has been emptied without escape of air.

Uses of the Chart

Example I. Assume a 20,000-gallon tank to be half full at mean operating pressure of 50 p.s.i. gage pressure. This condition is represented by a point at the intersection of the 50 per cent storage and 50 lb. pressure lines. Through this point draw a parallel diagonal; it will be about midway between the 15 and 20 lb. diagonal lines. As water is pumped in or withdrawn, the pressure-storage relations will be along this line. Thus, if water is pumped in until the pressure is 60 p.s.i. gage, storage will be 57 per cent of the tank capacity, and if water is withdrawn until pressure falls to 40 lbs., storage will be only 41 per cent of full capacity. It follows that effective storage within the pressure range of 40 to 60 p.s.i. is the difference. In this instance it is 16 per cent (57-41) of 20,000 = 3200 gallons in the operating range of 20 lbs. around a mean of 50 lbs.

Example II. Assume the tank and pressure range to be the same as in Example I, but that the operator wishes not to risk delivery of air to mains in case of power outage or failure of automatic pump controls. The diagonal line marked "O" applies to this condition. At 40 and 60 lbs. pressure, the tank will be filled to 72½ per cent and 80 per cent of full capacity at the respective pressures, and effective storage within this pressure range will be only 7½ per cent (80-72½) of 20,000 = 1500 gallons. Note that the low terminal pressure has been secured only by very substantial reduction in effective storage; this may not be objectionable if it does not cause too frequent operation of the pump. Or retaining the zero terminal pressure, more storage can be had by broadening the pressure setting.

Example III. Again considering the same tank and operating range of 20 lbs. around a mean of 50 lbs., maximum effective storage will be had when the tank is empty at the lower pressure setting. To provide a small margin of safety against delivery of air in normal operation, let us assume that the tank is conditioned for terminal pressure of 35 lbs. (5 lbs. below the pump starting pressure). On the diagonal for 35 lbs. taken as the zero storage axis, storage of 9 per cent and 33 per cent will be found at 40 and 60 lbs. pressure respectively. Effective storage is the difference (33-9), or 24 per cent of 20,000 = 4800 gallons. However, lacking an accessory device that would release the remaining air at a predetermined low water level in case the pump should fail to start, a conservative operator would probably prefer a condition more like that outlined in Example I or II wherein a lower storage valve is maintained.

Water Depth-Volume Relationship

There remains the problem of relating water volume to the level carried in the tank. The Depth of Fill-Capacity scale at the bottom of the chart is for this purpose. It does not take account of the capacity in dished heads, but the error will be negligible for practical purposes.

This scale has dual markings, the inner numerals (those nearest the scale) applying to tanks less than half full, the outer figures to those more than half full.

The H/D scale above the central line indicates the extent to which the tank is filled as a decimal fraction or per cent of its diameter. The numerals below the line show the corresponding actual storage as a percentage of the full capacity of the tank.

To illustrate, assume a tank of 20,000-gallons gross capacity, and 96 in. diameter. If 24 in. of water is pumped in, the H/D fraction is 24/96 = 0.25. Corresponding to 0.25 *above* the scale bar the actual storage shown below the bar is only 0.2 or 20 per cent of full capacity —i.e., 4000 gallons. Since the tank was less than half full, the inner numerals were used. Similarly, if it is desired to fill this tank to 70 per cent of gross capacity locate 0.7 *below* the scale bar and the corresponding value of .665 for H/D will be found in the *upper* line. That is to say the tank must be filled to 66.5 per cent of its depth or .665 x 96 = 64 in. (approximately) above the bottom. In this case the outer numerals are used.

Clarksburg Flood Hastens Stream Control Project

Since 1913 Scotland G. Highland, Manager and Chief Engineer for the Water Board of Clarksburg, W. Va., has been working on a plan to store 52 million gallons of water on the West Fork River Watershed, above Clarksburg. The chief purpose is flood control and, secondarily, an equalization of raw water quality at Clarksburg and other communities using West Fork water.

The U. S. Engineer's Office has made a study of the need for such flood control on the West Fork and Elk Creek above Clarksburg, and on March 5 issued notice that the project had been approved. Cost of the proposed dams and works is estimated at $14,050,000 and operating and maintenance cost at $35,000 per year.

Beside flood alleviation the control reservoirs will be bled during the dry weather periods to provide a water supply of lower hardness, acidity, iron and manganese content for downstream communities, the largest being Clarksburg, W. Va. In addition, the controlled release of water will provide increased dilution of sewage and industrial pollution of the West Fork, consequently a less expensive type of sewage and waste treatment may well be possible except in extreme dry years when the storage is exhausted before the drought is broken.

The flood of February 14, 1948, at Clarksburg proved the most serious experienced since July 9, 1888, according to Scotland Highland who reports that the water plant was kept in operation only by continuous sandbagging protection. This experience, with the most serious flood in 60 years, served to bring to a head the U. S. Army Engineer's decision to go ahead with the $14,000,000 flood, river stage, and water quality control project.

Rickard Now Superintendent of Water at Binghampton, N.Y.

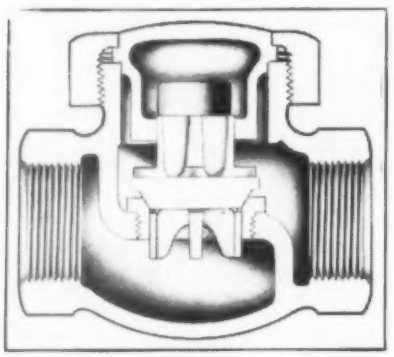
Grover E. Rickard, Superintendent of Water and Sewage Treatment at Oneonta, N.Y., since September 1942, has been appointed Superintendent of Water of Binghamton, N.Y., effective March 1. Mr. Rickard will continue to supervise the operation of the Oneonta Filtration Plant and Sewage Treatment Works on weekends until June 30.

Prior to his service at Oneonta, N.Y., Mr. Rickard was engineer in charge of filtration at Burlington, Vt., and for nine years prior to that was superintendent of filtration at Wheeling, W. Va.

CHECK VALVES, TRAFFIC COPS OF THE PIPE LINES

By H. J. BARTLETT
Manager, Alloy Sales,
CRANE CO.
CHICAGO, ILL.

CHECK valves are valves designed to permit flow in one direction only, and to close automatically if flow reverses. They are entirely automatic in action, depending solely upon pressure and velocity of flow within the line to perform their functions of opening and closing.



Lift Check Valve

Few people appreciate the real importance of check valves in a piping system, or the severity of the service to which they are exposed. Whenever a check valve is really needed in a line, it is just about the most important valve in the entire job. In the first place, it is distinctly a "safety first" valve. It is used either to prevent dangerous back flow in a line, or, in the case of two or more fluids being supplied to a common point at different pressures and different temperatures, to prevent one fluid from backing up into another line. Manually operated gate or globe valves could be used to control direction of flow, but instead check valves are used in order to obtain quicker, automatic action and extreme sensitiveness to changes in flow conditions.

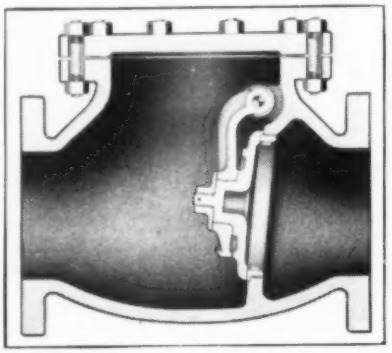
The fact that check valves are automatic in action renders proper selection even more important than for gate and globe valves. General experience indicates clearly that most automatic valves are subject to great abuse through the tendency of operators to rely on the automatic features performing without attention. Since the disc of a check valve in normal service moves with every fluctuation in flow and pressure, it follows that the moving parts of such a valve are subjected to much more severe wear than occurs in a manually operated valve.

Basic Types

There are only two basic types of check valves, the swing check and the lift check. In the straight swing check design, flow through the body is approximately the same as through a gate valve. A bridge wall, against which the disc seats, is transposed across the line of flow, and the disc is hinged to a pin placed crosswise of the body above the bridge wall. The disc is free to swing in an arc about this hinge pin, the lower limit of its travel being the bridge wall and the upper limit, some point on the body or cap far enough removed from the bridge wall to insure that full flow can take place. With no pressure or flow through the valve, the disc is held to the seat by gravity.

In the lift-type check-valve design, bodies of exactly the same design as regular globe and angle valves are commonly used. The disc is free to move in a straight line at right angles to the bridge wall and seating surface. To insure proper seating, the disc is equipped with integral guides above or below the seat or both. These valves are always installed so that the force of gravity holds the disc to its seat when there is no flow through the valve.

In a general sense, swing check valves predominate for liquid service, because liquids flow under low head pressures, or power is expended to pump them, and velocities are usually quite low. As a consequence, losses due to friction are important and extra precautions are taken to insure minimum resistance. For such service, swing check valves have a definite advantage, for their resistance to flow is much less than that of lift check valves.



Swing Check Valve

Furthermore, since the velocities are low in liquid lines, the destructive effect of "slamming" is greatly reduced.

Lift check valves are definitely considered to be superior to swing check valves for service on volatile and hard-to-hold liquids and on steam, air, gas, and vapors in general, provided there are no severe pulsations in flow. The deciding factors which influence choice in this case are that gases are usually much harder to hold than liquids and the velocities used are much higher. No shop difficulties are experienced in making lift check valves tight on such service, but swing checks offer a much more serious problem. Regrinding type swing checks can be made to pass an air test in the shop after special lapping, but a few operations usually cause them to leak again. The slamming of the disc in a swing check valve at both extremes of its movement is highly destructive, and this action is intensified as velocities in steam, air, and vapor lines are at least twenty times as great as in liquid lines. Knowing that even in liquid lines swing checks sometimes pound themselves to pieces, it is easy to imagine what happens to them when the velocities are so enormously increased.

The basic rule covering the installation of all check valves is that they must be applied so that the force of gravity tends to close the valve. The only exception to this rule is the case of the high-pressure vertical-type ball check valves which may be used in the horizontal position. Following this rule through, it will be seen that swing check valves can be used in horizontal lines and also for upward flow in any line inclined at any angle up to 90 degrees from the horizontal.

Swing check valves, regardless of the angle of the pipe line in which they are used, should always be installed with the hinge pins in a horizontal position. This insures that the disc will always swing in a vertical plane. There is sufficient play in the component parts of the hinge-disc assembly in swing checks to render imperfect seating likely if the valve is canted over to one side when installed.

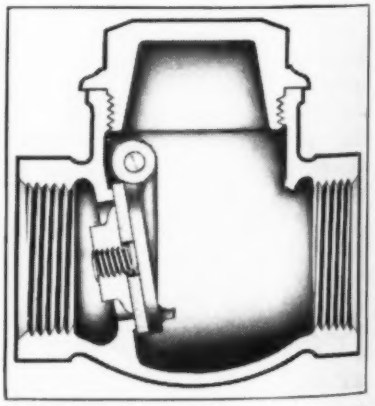
The same remarks, as far as alignment is concerned, apply to lift check valves. They should always be applied to a pipe line in such a way that the movement of the discs is in a vertical plane. This is particularly true of lift check valves with metal discs, for here, beveled seating surfaces are used which require that the discs be brought to the seats without canting or cocking if tight seating is expected.

Swing Check Valves

In the general family of swing check valves, there are a number of designs which merit some individual consideration, such as (1) common straight-way with metal disc, (2) common straight-way with regrindable disc, (3) straight-way with composition or leather disc, (4) angle type, (5) common foot valve, and (6) straight-way with outside lever and weight.

Common Straight-way Check with Metal Disc

Valves of this type have bridge walls with an inclination from the vertical of approximately 10 degrees.



Straight-way Check Valve with One-Piece Disc and Hinge

Since all swing check valves must be installed with the force of gravity tending to hold the disc to the seat, it is obvious that if the bridge wall were exactly vertical, the force holding the disc to the seat with no flow through the valve would be a minimum. Conversely, the force required to start raising the disc from its seat is also a minimum. Actually, such construction is impractical, because the small excess amount of power available to hold the disc to the seat renders the disc so sensitive to pulsations in flow that pounding and leakage are bound to occur. With the angle of inclination of the seat at about 10 degrees, there is sufficient gravitational force present to insure tight seating even on very low pressures. The increased angle of inclination has no appreciable effect on the frictional resistance to flow.

Some check valves of this type have a one-piece disc-and-hinge construction. There is no flexibility in the hinge-disc unit, and, therefore, if the hinge becomes bent through pounding or if slight body distortion takes place, the valve is bound to leak since the disc cannot find its seat. Replacement of the hinge-disc unit in these valves is also something of a problem, since great care must be exercised to insure that the seating surfaces are in proper alignment. Hence their use should be avoided where severe wear may take place. The valves are satisfactory for use on hot or cold water service and on very-low-pressure steam.

Common Straight-way Check with Regrindable Disc

The term "regrinding" is applied to those valves in which provision is made for regrinding the disc while the valve is in the line.

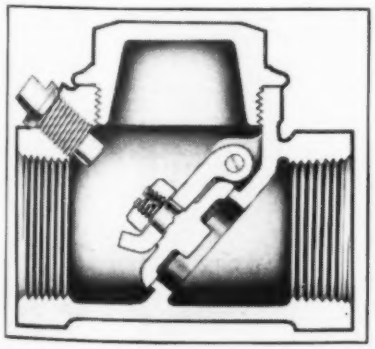
These valves have bridge walls inclined at 45 degrees from the vertical. The angle is chosen to provide a minimum overall length of body and yet allow for the provision of an opening in the body through which a tool can be inserted to revolve the disc on its seat. The resistance to flow in valves of this type is somewhat greater than in those having a more nearly vertical bridge.

Check valves of the regrinding type can be used on hot or cold water and on other fluids not corrosive to iron or brass. Like all swing check valves, they should not be used on pulsating flow. For steam service at all save low pressures, lift checks are preferred.

Straight-way Check with Composition or Leather Disc

These valves are similar to regular metal disc valves of the same type, but leather disc valves are intended solely for cold water service. There is a definite advantage in using soft disc valves, particularly where foreign matter may be present in the fluid being handled. On low differentials of pressure, the slightest particle of dirt on the seat of a metal disc check valve will hold the disc off the seat and cause leakage. With a soft disc this possibility is greatly reduced.

Making use of standard composition discs and disc holders of the conventional type insures that repairs can be quickly and easily made, and at a lower cost than for leather disc valves. As in all composition-disc valves, the seat is protected at the expense of the disc, which results in lower maintenance cost and greater life for the valve.



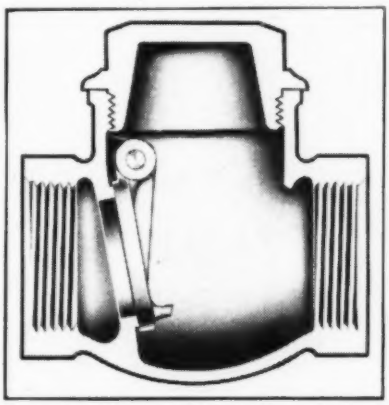
Straight-way Check Valve with Regrindable Disc

Common Foot Valves

Common foot valves have leather discs seating on the bridge wall, but the leather of the disc is extended beyond the seating surface to form an integral hinge which is anchored to the bridge wall. These valves are extremely simple in design, have a minimum number of parts, have no metal-to-metal contacts anywhere in their operating mechanisms, and give excellent service on dirty, gritty water. They cannot be used on hot water or on any fluid which will harden or attack leather.

Straight-way Swing Check with Outside Lever and Weight

Special features may be applied to ordinary swing check valves, for example, an outside lever and weight. In this design the hinge pin is rigidly attached to the hinge and rotates as the disc swings, the external lever moving through the same arc. The lever and weight can be applied to assist the disc in closing, thereby making the valve extremely sensitive to reversal in flow with immediate closure when forward flow ceases, or to balance the disc, causing the valve to open under a minimum of pressure.

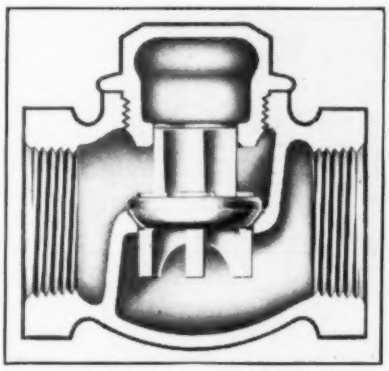


Straight-way Check Valve with Leather Disc

The first application (above) is frequently necessary where check valves are installed on the discharge side of centrifugal pumps operating against high heads. This is encountered frequently in water works service, in fire protection work, in mine pumping, and in general wherever water must be discharged to overhead tanks or elevated reservoirs. Centrifugal pumps deliver water with an absence of pulsations so that swing check valves are well adapted for this service. When high heads are involved, however, there is great danger of tremendous shock pressures being built up in the line when the pump shuts down and the check valve closes.

The extent of this shock pressure is far greater than normal maximum pressure in the line and is frequently great enough to fracture fittings, valves or other equipment not having inherently high shock resistance.

Study showed shock pressures were built up because the flow in the line reversed itself when the pump shut down, and this reversed flow assumed considerable proportions before the discharge check valve at the pump closed. The closing of the valve with reversed flow, already set up under a high head, was sufficient to cause a tremendous shock throughout the line. From this point on, the solution was simple. It was merely necessary to use a check valve which would close immediately when the pump stopped and before reversal of flow could start. This was accomplished very simply by equipping the ordinary swing check valve with an outside lever and weight in such a way that the weight aided in closing the disc. In actual service, such a valve will close when the pump stops, slightly before forward motion of the water ceases, and thus there is no reversal of flow at all.



Globe Type Check with Metal Disc

The second application of the lever and weight, where the device is used to balance the disc, is employed when valves are used on extremely low pressures, and particularly where valves are large, involving a very considerable amount of weight in the disc and hinge. Balancing the weight of disc and hinge results in easier operation and a less pressure drop through the valve.

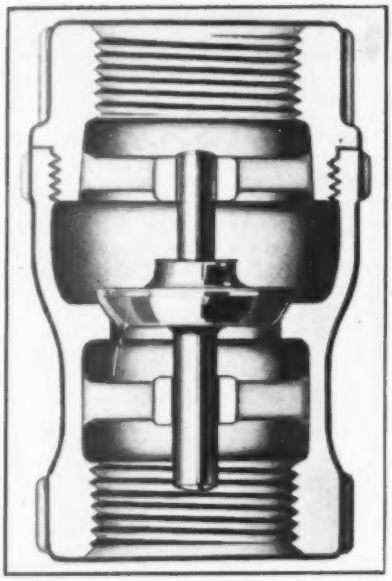
Lift Check Valves

Most lift check valves are definitely companion valves to globe and angle valves of the same pressure classification. The following classification will be used: (1) straight globe type with metal disc, (2) straight globe type with composition disc, (3) angle type, (4) vertical type, (5) foot valve, (6) ball type, (7) cushioned type, and (8) the stop-check.

Straight Globe Type with Metal Disc

This classification covers all metal-disc valves, both with and without renewable seats. Valves in this group are of the same general regrinding type, but there is a great variation in individual details of construction, including screwed caps, union caps, and bolted caps, depending on whether the basic globe valve had a screwed bonnet, a union bonnet, or a bolted bonnet.

A similar variation in design of guides for the discs may be found. All of these methods of guiding discs in themselves



Vertical Check Valve

alone are not sufficiently important to warrant acceptance or rejection of a valve. Accuracy of guiding is far more important than type of guiding.

The power available to seat the disc in any check valve is frequently very low. This means that unless there is perfect contact between the seat and disc, leakage occurs. For this reason, valves for air and gas should be specially ground and tested. In lift check valves, the disc and its guides are one solid unit. In the regrinding type, beveled seating surfaces are used, which require that the disc be brought to the seat without cocking. Accuracy in guiding is essential in order to insure tightness of seating surfaces under these conditions. Liberal area of guiding surfaces is an additional necessity if accuracy is to be maintained, for wear is accentuated in check valves due to the constant, and sometimes violent, movement of the discs.

Straight Globe Checks with Composition Discs

Composition disc check valves have several merits. Briefly, it may be said that for low pressures and on low differentials of pressure, it is easier to keep seating surfaces tight when soft discs are used. On liquids containing dirt or foreign matter, such valves offer a real advantage. Likewise on low pressure air, or gases and fluids which are hard to hold, composition discs are usually considered to provide tighter seating surfaces than metal discs.

For extremely low air or vapor pressures, soft disc valves, with a spring above the disc to aid in closing and maintaining tight seating surfaces, should be recommended. Air service in general is hard on check valves and it is advisable, therefore, to use sturdy valves for such service.

Vertical Check and Foot Valves

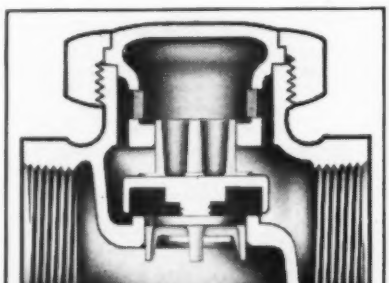
This group is the first type of lift check valve so far considered which is distinctive in body design. They are designed exclusively for upward flow, a service in which they meet competition from swing check valves. They differ from swing checks in that the disc moves in a straight line parallel to the flow, and is guided above and below the seats. The bridge wall containing the seating surface is interposed across the line of flow and at right angles to the long axis of the body of the valve.

Foot valves are essentially the same as vertical lift checks. In considering foot valves in general, it should be borne in mind that they are intended only for low-pressure service, since their purpose is merely to keep the suction pipe to a pump full in order to eliminate hand priming of the pump. Absolute tightness of seating surfaces is essential. For that reason, valves with soft discs are usually given the preference.

Construction of foot valves in general is quite light, as they are only intended to withstand a maximum pressure of 15 pounds. When centrifugal pumps are used discharging against high heads, foot-valves of the lift-check type should be avoided, for there is always the possibility of flow reversing when the pump is shut off, thus building up a pressure far beyond valve capacity. Use swing check valves if high pressures are likely to occur.

Ball Check Valve

This is a type of lift check in which a ball is substituted for the conventional guided disc. The principal advantage claimed for this design is that the ball never seats in the same position twice. This action tends to keep the seat clean. Likewise, the contact between the ball and its guides is very limited, reducing the possibility of sticking. To assure tight valves it is necessary that the ball be absolutely perfect.



Globe Type Check Valve with Composition Disc

Advancement in the art of manufacturing large size balls of extreme accuracy has been responsible for the development of satisfactory check valves of the ball type. In oil refinery service, where fluids are likely to leave gummy or coke-like deposits in valves, the ball check design has acquired considerable popularity because of its apparently better performance.

Cushioned Check Valves

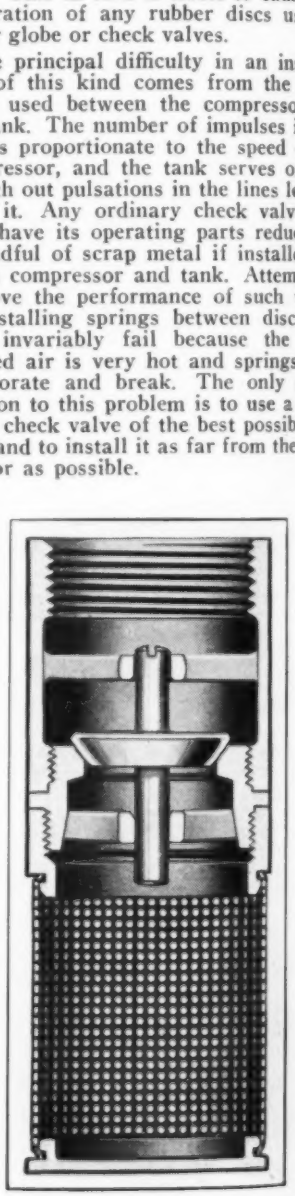
This type is worthy of special consideration. Cushioned check valves are all of the lift type. The upper part of the disc forms a piston operating in an accurately machined cylinder which may be an integral part of the body or a separate liner. The piston and cylinder forms an effective dashpot, which cushions the movement of the discs on both the opening and closing stroke and eliminates slamming and pounding.

For severely pulsating flow, the valves with friction rings are recommended because their cushioning effect is more pronounced than in valves with plain pistons. The latter are suitable where pulsations are only moderate in intensity or intermittent. Whenever trouble is being experienced with ordinary types of check valves going to pieces, it is well to remember cushion valves. They will indefinitely withstand severe service conditions under which any other type of valve will not last an even reasonable length of time.

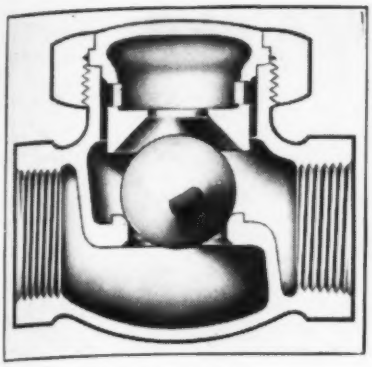
A good example of the need for a cushioned check valve may be found in the average air-compressor installation. Here we have a reciprocating compressor running at speeds from 200 to 500 revolutions per minute. Most such installations operate at pressures from 60 to 120 pounds per square inch. If the air storage tank is large enough, pulsations in the lines leading from the tank to the air-consuming apparatus are not usually severe and, if check valves are required, any good lift check should be satisfactory. If, however, the tank is too small, pulsations may be carried through into the discharge lines from the tank, in which case only cushioned check valves should be used.

Air-compressor storage tanks should be properly drained with an automatic trap, or oil from the compressor may be carried over into the discharge lines from the tank. This in turn is liable to cause disintegration of any rubber discs used in either globe or check valves.

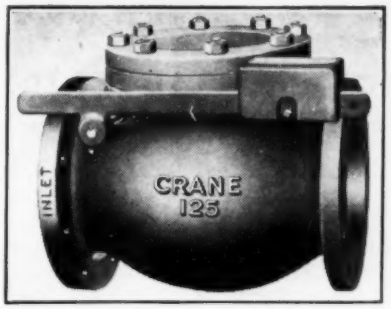
The principal difficulty in an installation of this kind comes from the check valve used between the compressor and the tank. The number of impulses in this line is proportionate to the speed of the compressor, and the tank serves only to smooth out pulsations in the lines leading from it. Any ordinary check valve will soon have its operating parts reduced to a handful of scrap metal if installed between compressor and tank. Attempts to improve the performance of such valves by installing springs between discs and caps invariably fail because the compressed air is very hot and springs soon deteriorate and break. The only sound solution to this problem is to use a cushioned check valve of the best possible design, and to install it as far from the compressor as possible.



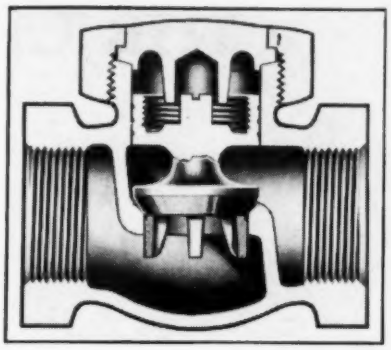
Foot Valve with Strainer



Ball Check Valve



Swing-check Valve with Outside Lever and Weights



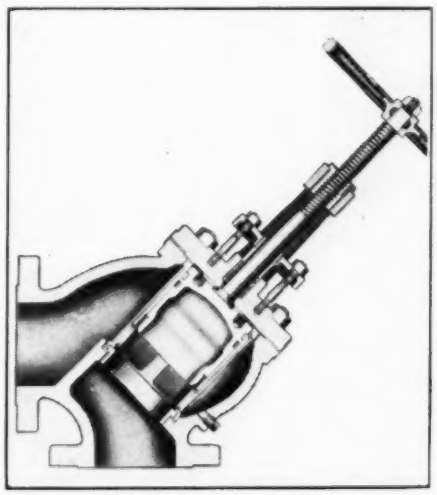
Cushioned Check Valve

Stop-Check Valves

This classification is a type of lift check in which external mechanical means are provided for seating the disc and holding it closed. In all of these valves, a threaded stem is used which has no mechanical connection with the disc. When the stem is screwed to the upper limit of its travel, the valve acts as an ordinary lift check valve since the stem then does not influence the movement of the disc. When screwed down to the other extreme of its travel, the stem engages the disc and forces it to its seat, and the valve then acts as a positive stop valve. By adjusting the stem to some intermediate point in its travel, the upward movement of the disc may be limited and the normal flow through the valve itself controlled. Upon reversal of flow, the valve will still act as a check valve.

For boiler feed service or any installation where pulsations in flow may be severe, it is considered advisable to insert a spring between the disc and the bonnet in order to prevent slamming of the disc, which might result in battering the end of the stem.

All moving parts operate in a removable liner which contains the seat, so that the entire mechanism can be renewed without taking the valve out of the line. The liner prevents expansion strains and distortion from being transmitted to the internal parts and thus eliminates binding and cramping, which frequently interfere with the free action of ordinary valves. The disc has a long throttling lip which prevents flow from starting through the valve until the disc is a considerable distance above the seat. Pounding, chattering and wire-drawing is thus reduced to a minimum.



Stop Check Valve

When it is desired to eliminate the stop feature, a plain flanged cap can be furnished in place of the usual bonnet, yoke, and stem.

These valves were designed specifically for service in steam lines between boilers and headers, although they perform admirably in many other locations in piping systems. An automatic stop-check valve should be installed in the pipe line from each boiler to the main steam header, when more than one boiler is connected to the same header. When so used, the valve performs several important functions: It acts as an automatic non-return and "safety first" valve, preventing a back flow of steam from the header into the boiler in the event of a failure or shutdown of the boiler. It acts as a manually operated stop-valve which may be closed when it is necessary to cut a boiler out of service temporarily without drawing the fire. A gate valve is usually installed at the header so that the line may be shut down when it is necessary to work on the stop-check valve. It automatically cuts the boiler into service when pressure has been built up to a point slightly above the header pressure, and conversely prevents it from going into service if it is not steaming properly.

Boston Civil Engineers Celebrate 100th Anniversary

The 100th Anniversary Meeting of the Boston Society of Civil Engineers was held on March 31 at the Boston City Club. The Boston Society of Civil Engineers, with a membership of 850, is the oldest engineering society in the country. Harvey B. Kinnison presided.

Following dinner in the evening, the Desmond FitzGerald Award was presented to Jack E. McKee, partner in the firm of Camp, Dresser & McKee. The Clemens Herschel Award went to Henry L. Kennedy of Sewey and Almy Chemical Co.; the Structural Section Award to Prof. Arthur Casagrande, professor of soil mechanics at Harvard University; the Transportation Section Award to Prof. Alexander J. Bone, associate professor of highway engineering at M.I.T.; the Hydraulic Section Award to Prof. Howard M. Turner, consulting engineer at Harvard University; and the Northeastern University Section Award to Leonard B. Loitherstein, student member at Northeastern University.

Speakers of the evening were Prof. Charles B. Breed, Dean Gordon M. Fair, Prof. Charles M. Spofford, Prof. Howard M. Turner, who collectively gave a "Centennial Symposium—Contributions of the Members of America's Oldest Engineering Society to 100 Years of Progress in Civil Engineering."

Professor Breed sketched the developments in transportation engineering, Dean Fair the contributions in sanitary engineering, Professor Spofford the progress in structural engineering, and Professor Turner the advancement in the field of hydraulic engineering.

New officers for the coming year are President, Frederic N. Weaver, Prof. Civil Engineering, Tufts College; Vice-Pres., Harrison B. Eddy, Jr., Partner, Metcalf & Eddy; Sec'y., Robert W. Moir, Senior Civil Engineer, Metropolitan District Commission; Treas., Herman G. Dresser, Partner, Camp, Dresser & McKee; and Directors, Chester J. Cinder, Senior Civil Engineer, Metropolitan District Commission, and Frederick S. Gibbs, President, Engineering Sales Corp.

M. R. Smith City Manager Lubbock, Texas

M. R. Smith, Jr., of Lubbock, Texas, a member of the firm of Parkhill, Smith and Cooper, Consulting Engineers, has resigned from the partnership to accept the appointment as City Manager of the City of Lubbock. The firm will continue as Parkhill and Cooper, located at 1006—14th Street, Lubbock, Texas.

THE CHEMISTRY OF WATER TREATMENT*

Part I. Coagulation

by A. P. BLACK
Professor of Chemistry
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IT IS well for those of us who are engaged in any phase of water treatment to occasionally review the basic principles having to do with the processes being employed. These articles are prepared with that in mind. They do not purport to contain any material which is new or original and no references to the literature will be cited. They have been prepared with the idea of briefly summarizing the most important chemical principles underlying certain processes of the chemical treatment of water. Part I will deal with coagulation, and Parts II and III with softening and corrosion control, respectively.

Coagulants may be defined as those substances which are capable of removing colloidal impurities from water, and coagulation is the process by which such removal is brought about.

As we look back over what has taken place during the past seventy-five years, it seems rather clear that the development of coagulation as an art was greatly influenced by the development of slow sand filters, the first of which was constructed in 1829 by James Simpson of the Chelsea Water Company, London. Its effectiveness for the purification of water is too well known to require discussion here. It was early noted that its efficiency was greatly improved after the formation on the surface of the sand of a slimy gelatinous coating which was and still is termed "Schmutzdecke." There seems little doubt but that when early American workers began their development of the rapid sand filter it was their desire to duplicate by chemical means this slimy coating or schmutzdecke on the sand.

Since a chemical was to be used, however, it had to possess certain qualifications. It must, first of all, be gelatinous in nature so as to resemble as nearly as possible the organic growths on the older slow sand filters. Furthermore, it must be non-toxic, it must be relatively inexpensive, and it must be capable of reacting either with water itself or with substances naturally present in water or which might be added to water so as to produce the necessary gelatinous precipitate. As we all know, aluminum sulfate or a filter alum was selected for study. Although many years have passed since alum was first used for this purpose, it is probably true to say that it is still the most widely used of all coagulants in water treatment.

It is the purpose of this paper to show that the remarkable properties of filter alum as a coagulant are not due to its gelatinous nature but to entirely different properties unknown to the earlier workers. It is one of the most remarkable coincidences in contemporary science that after the real mechanism of coagulation became known, filter alum still remained the most widely used of its class.

It is possible to write equations for the chemical reactions which take place when aluminum sulfate is added to water containing natural alkalinity or added alkalinity in the form of lime or soda ash. These are shown below:

1. $Al_2(SO_4)_3 \times H_2O + 3\ Ca(HCO_3)_2 = 2\ Al(OH)_3 + 3\ CaSO_4 + 6\ CO_2 + (X-6)H_2O$
2. $Al_2(SO_4)_3 + 3\ Ca(OH)_2 = 2\ Al(OH)_3 + 3\ CaSO_4$
3. $Al_2(SO_4)_3 + 3\ Na_2CO_3 + 6\ H_2O = 2\ Al(OH)_3 + 3\ Na_2SO_4 + 3\ CO_2$

While it is probable that there are cases where these reactions take place essentially as they are written, it is also probable that there are many cases where they do not go about it in exactly this manner. In order to understand that statement and the true mechanism of coagulation, it is necessary to examine the nature and properties of colloids.

Colloids, Their Classification and Properties.

The colloidal phase refers to any form of matter in a certain state of subdivision or, better, within certain limits of particle size. The unit of measurement to be used is the millimicron or mμ. This is one-millionth (0.000,001) of a millimeter, which is approximately one twenty-five millionth of an inch. While no definite limits may be set within which various phases of matter may be grouped, it is possible to state that the molecules and ions of true solutions usually vary in diameter from 0.1 to 1 mμ. The particles in the colloidal phase will usually fall between 1 mμ and 100 mμ; above the higher figure they merge into the suspensions and emulsions which may be separated more or less readily by physical means.

A solid or particle size between 1 and 100 mμ when dispersed in a liquid is called a suspensoid, and a liquid the droplets of which vary in diameter from 1 to 100 mμ when dispersed in another liquid is called an emulsoid. Both are colloids. In most respects colloids resemble true solutions; in ordinary lighting they are clear and transparent, they will not settle, and they may be filtered without removal of the colloidal material. Under proper lighting conditions, however, a bright beam of light passed through a colloidal solution produces what is called a "Tyndall Cone" which results from the reflection of part of the transmitted light by the tiny particles. Under the ultra-microscope colloidal particles are found to be in constant motion. This motion, termed the "Brownian movement," results from the bombardment of the tiny colloidal particles of the colloid by the rapidly moving molecules of the solvent. The boiling point, freezing point, osmotic pressure and vapor pressure of colloidal solutions are essentially identical with those of the pure solvent.

Colloids may be classified in a number of ways. In the first place, they may be classified as sols or gels. Sols are, of course, colloids which have the appearance of true solutions, whereas gels have a jelly-like structure from which the name colloid (Greek: Like glue) was derived. Sols and gels may be readily demonstrated by anyone by two simple experiments. For the first experiment, one takes two 1-liter beakers, one containing boiling distilled water and the other distilled water just above the freezing point. If to each beaker there is added two or three drops of a saturated solution of ferric chloride a most interesting contrast will be noted. The beaker containing the cold distilled water will exhibit only the very faintest color of the dissolved ferric chloride, hardly perceptible to any except the trained eye; the other beaker, however, assumes a deep wine-red color. The explanation of course is that at the temperature of boiling point the ferric chloride is hydrolized with the formation of colloidal ferric hydroxide and soluble hydrochloric acid. This ferric hydroxide sol is crystal clear and may be filtered through the finest laboratory filter paper unchanged. If the cold beaker be set aside for a few hours or over night, it will gradually color up until the same wine-red intensity is finally reached. If both are allowed to stand for a few days a reddish gelatinous precipitate of ferric hydroxide will result in each. This is due to the fact that other ions present in the solution will finally cause precipitation. If, however, these colloids are dialized and these foreign ions removed, the colloidal solution will be stable almost indefinitely. The writer has in his possession and has many times shown colloidal solutions of ferric hydroxide, of metallic silver and metallic gold, which are at the present time fifteen years old and which have maintained their strength essentially unchanged during that period.

Another simple experiment may be used to demonstrate the formation of a gel. In one beaker is placed 30 ml. of a clear saturated aqueous solution of calcium acetate. In a second beaker there is measured out 170 ml. of 95% ethyl alcohol. If now the alcohol is rapidly poured into the clear limpid solution of calcium acetate and then an attempt is made to pour the mixture back into the first beaker, it will be found that the mixture has set to a solid gel before the transfer can even be begun. This gel is so stiff that it may be penetrated by the finger only with difficulty, yet it is formed within a matter of moments from two clear free-flowing solutions.

Colloids may be classified as reversible and non-reversible. Ordinary starch solution is an excellent example of the former. If a little starch is added to boiling water and the mixture boiled for a few minutes, a clear, faintly bluish solution results. When allowed to cool it sets to what is termed "starch paste." However, if reheated it will again assume the appear-

*Condensed from lectures given at the Short School for Water and Sewage Works Operators offered jointly by the Georgia State Board of Health and the Georgia Institute of Technology, Atlanta, Georgia, in Sept. 1947.

ance of a solution. Egg albumin or ordinary egg white is an excellent example of a non-reversible colloid. A small amount of fresh white of egg added to water and thoroughly shaken yields what appears to be a true solution. If now this mixture is boiled the egg white is coagulated and it may not be again transformed to its original state since a permanent change in the material has taken place.

Colloids may be classified as lyophobic and lyophyllic. This classification has to do with the affinity of the colloidal particles for the dispersing medium. If the attraction between the colloidal particles and the dispersing medium is small, the colloid is termed lyophobic. If, on the other hand, the attraction between the colloidal particles and the dispersing medium is great the colloid is termed lyophillic. The viscosity of lyophobic colloids is usually about the same as that of the medium, and they are quite susceptible to coagulation. The viscosity of lyophillic colloids, on the other hand, is usually much greater than that of the pure medium. In water treatment we deal for the most part with lyophobic colloids.

Finally the most important of all, colloids may be classified as positive or negative since it may readily be shown that they bear electrical charges. There are a number of theories as to the source of this electrical charge, but these are beyond the scope of this discussion. It is sufficient for us to know that the particles of some colloids bear a positive charge whereas the particles of other colloids bear a negative charge. It is also true that colloids may be prepared in essentially neutral condition and it is further necessary to note that colloids may change their electrical charge from positive to negative or the reverse as a result of a change or changes in external conditions. The latter point is very important in the theory of coagulation since it has been shown by numerous workers that both aluminum hydroxide and ferric hydroxide flocs may be either positive or negative. This has a most important bearing, not only on floc formation, but also on the completeness with which a coagulant will do the job it is supposed to perform.

Methods of Precipitating Colloids.

Colloids may be precipitated in a number of ways. Three will be mentioned since each of these has found applications or plays a part in water treatment.

The first is by electro-phoresis. This rather involved word simply means the precipitation of a colloid of one sign on an electrically charged pole of opposite sign. As a specific example, if we immerse two electrodes in the beaker of ferric hydroxide sol whose preparation was described earlier in the paper and pass a direct current through the sol, the wine-red color may be observed to slowly drift toward the negative electrode and a precipitate of gelatinous ferric hydroxide forms on and below that electrode. Since the colloidal ferric hydroxide particles have moved to the negative pole we may assume that before they were neutralized they bore a positive electrical charge. There are, on the other hand, various colloids which when placed between charged electrodes will move to the positive pole. These, of course, are negative colloids.

It is interesting to remark at this point that the two types of impurities for which coagulation is most commonly employed, namely color and turbidity, are both present in water in the form of negative colloids. This point is emphasized since it is most important in the discussion to follow. During the past few years a number of devices employing this principle of electro-phoresis have been suggested for practical use under various trade names. They have not been particularly successful due to polarization of the electrodes and frequent cleaning which is necessary.

Colloids may be coagulated by other colloids of opposite sign. For example, if to a negatively charged colloidal sol there is added an equivalent quantity of a positively charged sol, something akin to neutralization takes place and a precipitate is formed which consists of the mixture of the two substances originally present. This is most important in connection with coagulation of a water and will be referred to again later.

In the third place, and most important of all, colloids may be precipitated by ions of opposite charge. For example, if to a stabilized ferric hydroxide sol bearing a positive charge a little hydrochloric acid is added, the sol will be precipitated by the negatively charged chloride ion. This is the explanation for the ultimate precipitation of the ferric hydroxide hydrosols whose preparation has been described. There is, however, a very great difference in the relative precipitating power of ions and this power results from the value of the charge carried by the ion. Keeping in mind that in a discussion of this sort, only very general statements can be made, it is possible to say that as a rule the precipitating power of a bivalent ion will be 50-60 times as great as a monovalent ion and the precipitating power of a trivalent ion will be 600-700 times as great as that of a monovalent ion.

It is this remarkable fact which throws most light on the action of soluble aluminum and ferric salts as coagulants in water treatment. It may be demonstrated by dividing a positively charged ferric hydroxide sol prepared as described above into three equal portions. To one there is added a small amount of sodium chloride. To the second is added an equivalent amount of sodium sulfate and to the third is added an equivalent amount of one of the soluble sodium ortho-phosphates. The sodium chloride will have very little effect in precipitating the sol, the effect of the sodium sulfate will be very much greater and that of the sodium phosphate greatest of all. If the experiment is carried on in such a manner that quantitative results will be obtained, values for the relative precipitating power of the ions approximating those just given will be obtained.

Having now reviewed some of the chemistry of colloids, we are in a position to examine the known facts of coagulation and to attempt to explain them in terms of the behavior of colloids. We have learned that both color and turbidity present in natural waters are present in the form of negative colloids. That leads us at once to the conclusion that for their coagulation we will need to employ either a positive ion or a positive colloid or both. As it happens both agents take place in the phenomenon of floc formation. We discover now by examining the chemical formulae for filter alum and ferric sulfate or ferric chloride that these coagulants each have a positive ion whose valence is three. Though chemists have carefully searched the periodic table of the elements and have made efforts to employ other elements of the same valence and elements of higher valence, it still remains true that these salts are most effective and most widely used as coagulants.

Mechanism of Coagulation and Floc Formation.

When a coagulant is added to water for the removal of color or turbidity three separate factors and three different types of action are involved. They will be discussed in the order in which they take place.

1. As the coagulant dissolves and the trivalent aluminum or ferric ions become available, they neutralize the negative charge on the particles of color or turbidity and, as we have seen, their neutralizing power is very great. This, in the opinion of most authorities, is the most important phase of coagulation and it takes place long before the visible formation of floc particles. It is evident that this action will be more efficient if the coagulant can be brought into intimate contact with the material to be removed before secondary reactions can take place. This clearly implies the desirability of rapid mixing as the first step in coagulation.

2. When the positively charged aluminum or ferric ions have neutralized a considerable portion, perhaps most of the negatively charged colloidal particles of color or turbidity, the resulting particles may be called micro-flocs for the purpose of this discussion since they are still beyond the limits of visibility and far too small to settle under the influence of gravity. Furthermore, it has been shown that, in the acid range of coagulation most commonly employed, they still retain positive charge, possibly due to the adsorption of positively charged hydrogen ions from the solution. We are now dealing, therefore, with a positively charged colloid which has the capacity to neutralize the negatively charged colloid. These positively charged micro-flocs can therefore continue to remove negatively charged particles of color or turbidity, and this constitutes the second phase of coagulation.

3. During both of the above phases no visible floc particles have formed. The mechanical treatment of the water from this point forward should consist of slow stirring, termed by some authors "conditioning" in order that these tiny micro-flocs may cluster together and agglomerate with the formation of floc particles which steadily grow in size until they are in a proper condition for sedimentation. During this phase of floc growth the third action, that of surface adsorption, takes place. It is outside the scope of this paper to deal with the various theories of adsorption, and the complex forces which may be and probably are involved. It is sufficient only to say that the amount of surface area exposed by the innumerable particles of floc is very great and that impurities in the water may be adsorbed on these active surfaces. It should be emphasized however that the formation of a large and well formed floc, while important from the standpoint of sedimentation, is not necessarily the criterion of successful coagulation since the most important functions of the coagulant take place, as has been shown, before the floc ever attains visible size.

Influence of Other Ions Present in Water.

It has been stated that the micro-floc particles formed by coagulation with either filter alum or ferric sulfate in the acid range bear a positive charge. This has an important implication to the water chemist since it should mean that a water

high in sulfate ion should be coagulated more readily than a water low in sulfate ion or high in chloride ion and that is true. While the precipitating power of the bivalent negative sulfate ion is not as great as of a trivalent ion it is still very high and it is true that sulfate waters are more readily coagulated in the acid range than are chloride waters, and for the same reason ferric or aluminum sulfate are better coagulants than the corresponding chlorides. It follows also that since the ions are charged, each ion present in a given water will have some effect on floc formation, the positive ions in the water interfering with the formation of a positive floc and the negative ions helping in its formation. This is one of the important reasons which explains the individuality of waters as far as their coagulation is concerned. As will be shown in the next paper in this series, the softening of water by any of the known processes follows well defined chemical laws and dosages, and results can be calculated in advance. This is not true with coagulation. The first of the chemical equations which has been listed above presents what would happen if filter alum reacts completely with natural alkalinity present in a water. Assuming for filter alum a "practical" molecular weight of 600, which corresponds to about 14.5 molecules of water of crystallization, one grain per gallon of alum would reduce the alkalinity of a given water by 8.5 parts per million. If pure aluminum sulfate whose formula is $Al_2(SO_4)_3$ .18 H_2O, and whose molecular weight is 666 be used, then the theoretical reduction in alkalinity by one grain of alum is 7.7 parts per million. Every water works operator who is experienced in coagulation is, however, well familiar with the fact that it is possible to add a considerable dosage of alum to a water with alkalinity reduction substantially less than called for by either of these figures. This indicates the correctness of the theories of coagulation which have been set forth above and results from the fact that a considerable fraction of the aluminum ions instead of reacting with the alkalinity of the water react directly with the negatively charged particles of color or turbidity.

In the case of organic color, the flocs formed are called "color flocs" and are quite different in properties and appearance to the gelatinous aluminum hyroxide formed by the reaction of aluminum sulfate with alkalinity in water. Although this is true, it has not been found possible to arrive at equations for calculating the amount, which will hold in every case, for a given coagulant to remove a given amount of color or turbidity from a water. Such equations are published from time to time and are usually valid for the particular water from whose treatment they are derived but as a rule they are found not to hold in case of other waters. This can only mean of course that we do not as yet have sufficient information with regard to the true nature of the colloidal impurities which we are removing from water. This is particularly true with respect to organic color. After more than fifty years of water treatment, we still find ourselves in the position of using more or less rule-of-thumb methods for the removal from water of a material about whose chemical composition we know relatively little. Much research needs to be done on the fundamental nature of color in water.

Coagulants Used in Water Treatment.

It is hardly worthwhile to attempt a complete listing of the coagulants which have been used in water treatment since most water works men are familiar with them and they receive frequent mention in the literature. As has been previously stated, aluminum sulfate continues to be the most widely used material of its kind but there appears to be an increasing tendency toward the use of ferric sulfate. Sodium aluminate often finds application, particularly in industrial water treatment. Ferric chloride and chlorinated copperas have been used and in the Middle West a combination of ferrous sulfate and lime is widely used, mainly for the removal of turbidity. It was one of the first coagulants to be used in this country, having been used by Bull in Quincy, Illinois, in 1898.

The use of Bentonite has been suggested but it has not been generally adopted although it has proved useful in certain cases. The use of activated silica as an accessory coagulant deserves special mention. A solution of sodium silicate, activated with sulfuric acid as recommended by Bayliss, by aluminum sulfate as recommended by Graf and Schworm, or by ammonium sulfate, chlorine, carbon dioxide, and so forth as recommended by Hay and others, yields a stable sol bearing a high negative charge. In the acid range it effectively promotes coagulation and floc formation with either aluminum sulfate or ferric sulfate. It is not generally recognized, however, that activated silica may act as a coagulant in the truest sense of the word since, in softening many waters by the lime-soda process it results in the formation of a better-formed and better-settling sludge than either alum or ferric sulfate. The new 80-million gallon softening plant of the City of Miami is being designed to employ activated silica as the only coagulant although facilities will be provided for feeding alum to anticipate possible changes in water quality. In the case of this particular water a dosage of 2-4 parts per million of activated silica yields better results than a dosage of three grains per gallon of alum. While activated silica used in conjunction with another coagulant should properly be called a coagulant aid, the time has come to recognize that it may and often does act as a coagulant in the truest sense of the word.

Laboratory Control of Coagulation.

The most important single tool at the disposal of the chemist in charge of the operation of a coagulation plant is the jar test machine and there is no substitute for numerous and properly conducted jar tests for the control of such plants. Only by such test can the most economical dosage of coagulant be arrived at and the best settling flocs be obtained. There are many excellent papers in the literature describing the proper technique for such tests and they are also described in several of the contemporary text books and manuals. Every operator who does not have a jar test machine and is not using this test, should consult these references and institute such tests if he desires to obtain best results from his plant. Along with the jar test it is necessary to make regular and periodic determinations of color or turbidity, alkalinity, total hardness and pH value. The latter is especially important since it has to do with the stability of the treated water. In the case of most so-called "Filter" plants which employ coagulation, the treated effluent is apt to be quite soft and of relatively low pH value. There are a number of ways by which such waters may be stabilized and that subject will be discussed in one of the other papers which are to follow.

The ever-increasing popularity of the newer types of up-flow equipment emphasizes the need for a better understanding of coagulation since retention times are far less in these types of units than in the older and more conventional settling basins. For the same reason a great deal more vigilance and careful supervision is indicated in the operation of such plants. It is doubtful if anywhere in American chemical industry there is to be found a comparable example of the use of chemicals in the tonnages consumed by the water works industry and in the dilutions employed by it. The chemical reactions, not only of coagulation but of softening as well, are carried out in extremely low concentrations, and it is a tribute to the ingenuity, resourcefulness and scientific ability of the water works man that they are so successfully employed under these unusual conditions.

Cast Iron Pipe Still Short

According to the February issue of *Public Construction,* released by the Federal Works Administration of Washington, D.C., Cast Iron Pressure Pipe is still short. This appears to be indicated by the continued increase in unfilled orders which rose in November to 965,460 tons, 3 per cent above the October figure, and 30 per cent above the November 1946 figure. Shipments in November declined to 81,061 tons, a decrease of 19 per cent from that shipped in October, 1947. At the present rate of production unfilled orders now equal over 11 months of current production.

Black Laboratories, Inc.

Near the end of February occurred the formal opening of the Black Laboratories, Inc., 968 S. Oak Street, Gainesville, Florida. This firm, which will specialize in water and sewage and the production of standard solutions and indicators, will have as its President, Charles A. Black, son of Dr. A. P. Black, Professor of Chemistry at the University of Florida. Secretary-Treasurer of the organization is William B. Crow of Gainesville. Both are graduates of the University of Florida, and have been connected with the City of Gainesville for several years, Mr. Black as City Chemist, Mr. Crow as Ass't Supt. Municipal water and light plant. Dr. A. P. Black will also be associated with the firm as Consulting Chemical Engineer.

N.J. SEWAGE WKS. ASSN'S. 33RD ANNUAL MEETING

Honors Harry P. Croft; Features Session on Industrial Wastes

THE thirty-third annual meeting of the N. J. Sew. Wks. Assn. was dedicated as a testimonial to Harry P. Croft, Chief Engineer of the N. J. Dept. of Health. The occasion for the honor was the thirty-fifth anniversary of his association with that department, and the thirtieth anniversary as Chief Engineer.

Presidents

(Retiring)
L. J. Fontenelli
Supt.
Treatment Wks.
Rahway

(Incoming)
L. E. West
Chief Engr.
Jt. Meeting
Elizabeth

As a part of the ceremony, Sam Probasco, City Engineer of Burlington, N.J., presented Mr. Croft with a hand lettered scroll containing the names of his many friends and admirers. Also presented to Mr. Croft was a beautiful television, phonograph, and radio console. Mr. Croft was a charter member of the association and has been active in New Jersey in the attempt to raise the status of the sewage treatment works operator in that state.

Business Session

At the annual business meeting the following officers were elected to serve for the ensuing two year term:

President

Leslie E. West, Chief Engineer.
Joint Meeting Sewage Works,
Elizabeth, N. J.

1st Vice President

John Struss, Superintendent.
Sewage Treatment Works,
N. Wildwood, N. J.

2nd Vice President

Sol Seid, Superintendent.
Sewage Treatment Works,
New Brunswick, N. J.

Secretary-Treasurer

M. S. Kachorsky, Superintendent.
Manville, N. J.

Member Fed. Bd. of Control

L. J. Fontenelli, Supt. Treatment.
Garwood Sew. Treatment Works,
Rahway, N. J.

Harry P. Croft was elected to serve on the Executive Committee of the Association, and the constitution of the association was changed to include the past president on the executive committee which automatically made the retiring president, L. J. Fontenelli, a member for the next two years.

Honored
Harry P. Croft
Chief Engineer
State Dept. of Health
Trenton

"A friend of the sewage plant operator." 35 yrs. with the State Health Dept.—30 yrs. as Chief Engr.

Stream Pollution

"The Effects of Stream Pollution on Industrial Expansion and Development in New Jersey" was the title of the paper presented by **T. Ledyard Blakeman,** Chief, Bureau of Planning, New Jersey Department of Economic Development. Mr. Blakeman, who pointed out that planners work at least five years in advance, said that the legal basis for pollution abatement is the protection of health and in a very narrow sense, the general welfare of the people. In the last analysis the reasons for preventing further pollution of streams is to do what is possible to ameliorate the unsightly and costly conditions which reduce the desirability of New Jersey as a place to live.

Vice-Presidents

(First)
John Struss
Supt.
Sewage Wks.
N. Wildwood

(Second)
Sol Seid
Supt.
Treatment Wks.
New Brunswick

Mr. Blakeman listed five major deterrents to development of New Jersey: These are lack of research, planning and promotion comparable to other states; poor transportation into New York and Philadelphia; high municipal taxes; limited ground water supplies; and increasing pollution problems in the industrial areas.

In conclusion the speaker said that the solution of the industrial waste

S.R.O. Session and Exhibit Booths

(1) Lou Frazza explaining compression joint for Transite sewer pipe. (2) Kopper's booth featuring W.D.H. Bitumastic coatings. (3) Packed session for the Croft testimonial ceremony. (4) In Chicago Pump's booth "Doc" Symons pops some questions to C.P.'s Frank Roe. (5) W. & T's. demonstration of their new Economy Program Sewage Chlorinator.

Programmers

J. K. Adams, Supt., Tenafly — *Lester Pope, Operator, Hamilton Twshp.* — *E. T. Killam, Cons. Engr., New York* — *Paul Molitor, Supt., Rutherford* — *R. E. Bonyun, Genl. Supt., Passaic V. Comm.*

problem in New Jersey is not the responsibility of any single group; it is of vital concern to the whole population. The state and the municipalities therein are faced with a choice of public expenditure to compensate for past mistakes and to guard against them in the future or a decline in population and productive capacity will result.

First of the discussers of this paper was **W. B. Hart,** Director of Waste Control, The Atlantic Refinery Co., Philadelphia. Mr. Hart referred to the growing interest in surface waters because of the failing ground water supplies, and pointed out that industry is about one-third of the way towards full realization of the problems of stream pollution.

New Jersey has entered the program of stream pollution abatement. In the last three years nearly forty states have strengthened their laws for surface water protection, and many of these states have made appropriations for stream control. This means that something is going to happen in the future and that stream pollution abatement will progress more rapidly than it has in the past.

Industries, which deal in actualities, want more than indefinite terms as to treatment. They need to fully understand the terms used, and would like to have definite standards, not just a statement that equivalents of primary treatment are desired.

Mr. Hart said that if a stream could serve these purposes, fish, industrial water supply, and domestic water supply, it is sufficiently free of industrial waste. He suggested that standards should be based on the biological condition of the stream rather than on the chemical analysis of the water.

Dr. Harry W. Gehm, Tech. Dir., National Council for Stream Improvement, said that only five per cent of the total paper products of the country are manufactured in New Jersey, but there is a sizable box and container board industry. In sixteen mills varying in capacity from 30 to 375 tons, a total of 2200 tons of board are produced each day, amounting to 12 per cent of the country's total of that material. On the other hand only 375 tons of paper are manufactured daily in twelve mills. About a third of the waste is treated in municipal sewage plants and the other two-thirds goes into streams.

"M.C."

Sam'l R. Probasco
City Engr.
Burlington, N.J.
(Principal Speaker at Croft Testimonial.)

Dr. Gehm stated that for the most part municipal treatment works can handle this type of waste. For normal conditions, sedimentation and filtration to remove suspended matter is sufficient treatment of the waste, but lack of clean used-paper has so increased the strength of waste that pollution problems have been intensified. As a result the National Council for Stream Improvement has instituted research at Rutgers and elsewhere to develop treatment processes.

John G. Segelken, Special Representative, Bell Telephone Laboratories, told of the studies being made in the Bell Laboratories and said that the wastes from these experimental projects constitute 10 per cent of the total waste from the Laboratories. Plans are underway to install holding tanks and thus maintain a constant rate of discharge of the waste to a municipal sewage treatment works.

E. I. Merrill, Plant Engineer, Johns-Manville Corp., reviewed the problem that once existed at the J-M plant and told of the steps taken to reduce the pollution of the Raritan River. In addition to reducing stream pollution the steps taken also conserved materials, heat, and water, thus reducing operation costs. The program has taken 13 years and was developed on a dollar and cents basis. Segregation of wastes, recycling of waters, use of vacuum filters, and passage of water containing suspended solids through three sedimentation basins has been effective treatment. The sedimentation basins, in series, have retention times of 22, 23, and 48 hours, and suspended solids concentrations of 700 to 1200 ppm. in the raw waste are reduced to 50 ppm. in the effluent. One particular problem of a strongly alkaline waste was solved by piping it to a neighboring chemical plant having an acid waste requiring neutralization before discharge to the stream.

B. W. Dickerson, Engineer, Hercules Powder Co. Inc., Wilmington,

Bill Johnson, Pitts. Equit. Meter, New York, and Roy Evans, Mueller Co., New York

Harold Eifler, Supt., Totowa, N.J., and John Wichert, Supt., Butler, N.J.

Sam Probasco displaying handsome Testimonial Scroll to Harry Croft (left) who was also presented a Television Set.

C. B. McMinamin, Supt., Middlesex, N.J. **and** *Louis Staffelli, Mayor of Middlesex.*

D. Cummings, Operator, Ridgewood, N.J., and Chas. Lyons, Supt., Fairlawn, N.J.

Del., pointed out industry's interest in uncontaminated waters, because polluted waters require purification, and water for processes needs a high degree of treatment. Conservation of water supplies and prevention of pollution is important to industry because it requires so much water of which there is a short supply.

If the streams are cleaned up there will be enough water for all uses. To overload a stream with pollutants destroys the value of the stream and the surrounding area. Dickerson discussed the need for higher treatment of industrial wastes, methods of treatment, and suggested the need for pilot plant studies, carried on preferably by state departments of health.

F. S. Friel, Cons. Engr., Philadelphia, summarized the discussion of the various speakers and pointed out that there is no broad corrective program as yet developed. Conditions such as exist today cannot be continued without penalty to all interested parties according to Mr. Friel. It is apparent that industrial waste problems are not all solved yet, but that industries are cooperating, and will need to continue to cooperate as will all scientists in the field of waste disposal. Mr. Friel emphasized the fact that waste treatment costs must be considered as a legitimate part of process production costs, and he recommended that industry look to the Federation of Sewage Works Assns. for help, guidance and cooperation in solving industrial waste problems.

Sewer Authorities

In regard to "The Establishment of Beverly Sewerage Authority," **G. Joseph Devine,** Chairman of the Authority, said it was an absolute necessity in order to handle both the domestic and industrial waste problem in that community. Other methods for setting up a sewage treatment works did not appear to be feasible whereas the establishment of a Sewer Authority was both feasible and practical. Principal industrial waste is from a dye mill. Installation of pre-treatment and equalizing tanks at the dye works and payment for a fair share of the cost of operation appears to be the solution of the waste problem of Beverly. Treatment will include both primary and secondary processes. Schedules of charges have been set up on the basis of number of units of plumbing fixtures.

Programmers

W. B. Hart, Engr., AtlanticRefining, Philadelphia

E. I. Merrill, Plant Engr., Johns-Manville, Manville, N.J.

I. R. Ricker, Boro Engr., Princeton, New Jersey

H. W. Gehm, Tech. Advisor, N.C.S.I., New York

Frank C. Roe, Eastern Mgr., Chicago Pump, New York

Chairman

John J. Horgan, Distr. Mgr., Ralph B. Carter Co., Hackensack, N.J. (Croft Testimonial Comm. Chairman)

Alexander Batcho, Mayor, Manville, N.J., described the situation in his municipality and why it established a sewer authority. Principal reasons were the construction of a Federal Housing Unit which increased the load on the plant by 25 per cent but paid only 4 per cent of the cost of operation, and secondly because of the city debt structure. By ordinance, the city turned the sewage works over to the Authority which assumes the bonded indebtedness of the sewer system and plant. Mr. Batcho said it was important to remember that whether or not a municipality needs more borrowing power, a sewer authority is the fairest system for the users.

Sewer Rentals

In its program to equitably distribute the cost of operation of its sewage treatment works, the city of New Brunswick, N.J., has taken its second step according to Hon. **Chester W. Paulus,** Mayor. The second step was the establishment of sewer rentals by ordinance. The first step which was initiated in 1944 was the establishment of industrial waste charges. The establishment of sewer rentals gets around tax exempt properties, makes it possible to charge industries for service outside of the corporate limits, and assesses a charge for sanitary waste from industries as well as the industrial waste. Charges are based on water consumption. Where the user has a private water supply, it is metered. Corrections are made for water used but not discharged to the sewers. The minimum charge is $10 per year which is two-thirds of the minimum water bill.

Controlling Infiltration

Under the above title for his paper, **I. Russell Riker,** Borough En-

Ed. P. Molitor, Supt., Chatham, N.J. and Clyde Molitor, Asst. Supt., Chatham, N.J.

Jack Collum, Supt. Public Wks., Bordentown, N.J.

Harry P. Croft

"Bob" Mowry, W. & T. Co.

Dorr's Booth (Left to right) John Baffa, Cons. Engr. Alan Darby, Dorr Co. "Doc" Fischer, Dorr Co. and "friend"

Fred Hugo, Secretary and Paul Molitor, Supt., Joint Meeting, Rutherford, N.J.

Frank Gerlitz, Simplex Valve & M.

J. M. Angell, Jr. "Water & Sewage Wks."

Frank C. Roe, Chicago Pump

gineer, Princeton, N.J., discussed the problems of excess water in sewers and suggested means for controlling it. Often, 30 to 50 per cent of the total flow in sewers is infiltration, and under certain conditions the infiltration may be ten times the normal dry weather flow. In order to provide for this excess, water plants and sewers must be designed larger than might be expected. In one case, where the treatment works cost $350,000 the plant had to be made one-third larger than would have been necessary had there been no infiltration existing. The amortization costs plus the increase in pumping costs to handle this water amounted to $14,000 a year.

Mr. Riker had six suggestions for a program to control infiltration. These six points included:

1. City ordinance to prohibit rain water leaders, cellar drains, and cooling water from entering the system.
2. In existing system where the ordinance cannot be retroactive, eliminate bad conditions by education.
3. Make a survey of the system, inspecting manholes for leaks.
4. For new sewers, design to prevent infiltration by providing underdrains with gravity outlets in wet ground or where seasonal high water tables are common, and make specifications specific about backfill, firm foundation, good joints, and a test for tightness.
5. Insist on an inspector on the job at all times for new work.
6. Make tests for leakage before accepting a new sewer, and make these tests in wet weather, not in the dry weather season. Hydrostatic tests are best.

In his paper Mr. Riker also discussed storm water, existing leakage, rainfall versus sewage flow, precautions in laying new sewers, hydrostatic tests, and house connections. Mr. Riker discussed the infiltration survey at Princeton in some detail and emphasized the fact that technical articles on the subject of infiltration have not been very plentiful. In fact, few cities have adequate data on the subject.

Federation Affairs

In a short talk on "Current Federation Affairs," **W. H. Wisely,** Exec. Secy., Federation of Sew. Wks. Assns., told of the growing membership, now on the road to 5,000. Services of the Federation continue to grow as well, and there are now nine manuals of practice under development, with three nearing completion. (*The Glossary of Terms* will probably be first). The Federation research program is being carried on in six laboratories at present, and for the next year a sum of $40,000 has been asked from the Public Health Service to carry on research projects at seven institutions in 1948-49.

Speakers

(Sewerage Financing)
Hon. Chester W. Paulus, Mayor
New Brunswick, N.J.

G. Joseph Devine, Chairman,
Sewerage Authority of Beverly, N.J.

Hon. Alexander Batcho, Mayor
Manville, N.J.

One item of interest to all members is the increased cost of publishing *Sew. Wks. Jour.*, now up to $20,000 a year from a cost of $12,000 a year before the war. Lack of space and paper has made it impossible to publish articles immediately; some are now scheduled for three issues in advance and because it is a bimonthly publication articles presented at this meeting will not appear until the September issue or later.

PLANT OPERATORS' SYMPOSIUM

One session of the meeting was devoted entirely to plant operating problems. Leading off the discussions was **I. O. Lacy,** Supt. Sewage Treatment, Lockport, N.Y. Mr. Lacy's highly amusing presentation of some of the many operating problems he has encountered was titled, "Digitalis and Splints for Digesters." Part of the problems stem from the fact that Lockport is built on the famous Niagara Escarpment on two levels, one at 490 feet above sea level and the other at 640 feet. This tremendous fall in some of the sewers causes the sewage solids to be very finely divided when reaching the treatment works. The plant itself is built on soils deposited in the ancient falls bed. As a result settling of the digesters caused cracking, binding of the scum breakers, etc. Oil in the digesters from industrial wastes, lack of gas for heating, copper which killed digestion, all contributed to difficulties. Fortunately lagoon space has been handy and sludge from at least one digester has been lagooned regularly. In fact, in seven years of operation, it has not been possible to have all of the plant equipment working at one time. Most of the troubles have been mechanical failure due to settling of the digesters, although copper, tomato seeds, and oil have played a part in preventing proper operation.

At Hamilton Township, **Lester Pope,** Operator, said his worst trouble was sticking of lubricated plug valves because they had not been properly lubricated. When properly cared for no troubles ensued. One other problem at this plant was poor chlorinator operation during cold weather. This difficulty was solved by installing a Humphreys Gas Heater in the chlorinator building.

At Tenafly, N.J., **J. K. Adams,** Supt., solved the problem of brass corrosion in vacuum filters (activated sludge filtration) by installing Saran plastic tubing. Saran costs about the same as brass but is unaffected by a pH of 5.0. Screen plugging is one of the problems brought about by the use of effluent water for chlorinators. In this plant these screens are cleaned by bristle brushes; it has been found that plastic bristles last

L. F. Frazza
Johns-Manville
New York

Henry Raft,
Plant Supt.
New York

B. I. Corson
Teacher
Oaklyn, N.J.

R. B. Mowry
W. & T. Co.
Philadelphia

Colvin Ellis
Supt.
Moorestown, N.J.

Raymond Case
Supt., Merchantville, N.J.

Ed. P. Molitor
Supt.
Chatham, N.J.

H. Van der Vliet, Supt., Maywood, N.J. — *C. Clendenny, Supt., Westwood, N.J.* — *Hal. Hutton, W. & T. Co., Newark, N.J.* — *Ed. P. Decher, Sec'y, Jt. M't'g., Elizabeth, N.J.* — *A. J. Fischer, Door Co., New York*

four to five times as long as hog bristles.

Sludge from the Tenafly plant is dried and sold as Tenafly Soil Food at $30 a ton. Last year these sales netted a profit of $4,000.

Fort Dix, N.J., has some unusual operating problems and to obtain 90 per cent removals has to resort to unorthodox operation according to **A. J. Lanning,** Santary Engineer, First Army. It was found that removals dropped off every four to eight months, and the reason appeared to be the collection of solids in corners of the primary settling units. Continuous operation of sludge removal scrapers and cleaning of pockets of sludge is necessary to maintain a continuous efficient operation. Grease which should be removed in grease traps but isn't caused difficulties but these have been overcome by discharging all skimmings to old Doten tanks constructed during World War I. Supernatant liquor from the digesters is also taken to the Doten tanks, the overflow from which goes to a slow sand bed. Sludge digestion is carried on in three stages, with highly successful results. Mr. Lanning recommended that treatment of supernatant liquors from digesters is needed to eliminate some of the headaches of sewage works operation.

Industrial wastes in a sewage treatment plant may completely change the normal operation practice according to **Sol Seid,** Supt., Sewage Treatment, New Brunswick, N.J. Quantity, frequency of discharge, and quality of the wastes all have a bearing on plant operation. Breakdowns of industrial plants, unauthorized dumps, wide fluctuations in pH are important factors which affect plant operation. Particularly is this true in the case of a treatment plant using chemical precipitation. In one case in New Brunswick, the sludge load of a single industry is equivalent to a domestic population of 100,000 persons. This load made necessary the expanding of the sludge handling facilities. Because of certain industrial wastes present in New Brunswick sewage it is possible to operate the chemical coagulation without the use of ferric chloride. Only lime is used. Not all of the problems of industrial waste in New Brunswick are headaches. Industry has been cooperative and in many instances objectional wastes have been eliminated. A charge for industrial waste equalizes some of the problems, as well as paying for the cost of treatment of these strong wastes.

Secretaries

(N.J.S.W.A.) M. S. Kachorsky, Supt., Manville, N.J. — *(W.S.W.M.A.) Arthur T. Clark, Mfgrs. Ass'n, New York*

In the treatment works at Maywood, N.J., grit is one of the causes of operating difficulties according to **Henry Van Der Vliet,** Superintendent. All grit was removed by hand at first but mechanical removal helped solve the difficulties. Two chemical wastes in Maywood cause trouble. One was a chlorinous waste, strong enough to cause a residual chlorine in the sewage of 200 ppm., the other waste was from a cocoa plant and extremely high in B.O.D. These wastes were eliminated by a cooperative study with the industry which was able to recover a valuable by-product.

Using old Imhoff tanks for secondary settling tanks was not successful. Solids rose in the gas vents, and it was necessary to maintain a continuous spray in the vents to keep the sludge down. It is planned to install a conventional clarifier.

Economics of Sewage Treatment

Speaking on the subject of "The Economics of Sewage Treatment Plant Operation," **Leslie E. West,** Chief Engr. Joint Meeting Sewage Works, Elizabeth, N.J., pointed out that the larger the plant, the cheaper the cost per unit operation. The joint meeting is fairly large, treating a population tributary of 365,000 persons for $142,000 per year or 39 cents per cap. per year. Costs of construction are up; so, too, are labor, maintenance, and replacements. Mr. West has found that whereas construction costs have increased from an index of 210 (E.N.R.) before the war to 437 at present, the cost of operation index when figured in the same way has only gone up to 364 at present. It is still rising, but Mr. West believes it may be levelling off somewhat. In his own plant, Mr. West has found that insurance costs are up 36 per cent, fuel oil 43 per cent, and labor 42.6 per cent. Barging of sludge to sea has only increased by 5.5 per cent but future contracts are expected to be higher.

Paul Molitor, Supt., Joint Meeting at Rutherford, N.J., suggested that these increases in the cost of operation were causing an increase in the cost of ingenuity, developing a closer relationship between the operator and the community, and increasing the neighborliness of operators in the loan or exchange of equipment, services, and ideas.

Elson T. Killam, Consulting Engr., New York City, pointed out that prior to 1930 there was little mechanical equipment in sewage

W. E. Collings, Supt., Collingswood — *Jim Moncrieff, Diamond Alk., Philadelphia* — *Sol. Seid, Supt., New Brunswick* — *Frank Mowrey, Operator, Barrington* — *Sam Cenicola, Supt., So. Hackensack*

treatment works, but since that time much has been installed. Much of that which was installed in the thirties is now wearing out and costs for maintenance or replacement are higher than the original costs.

Mr. Killam believes that municipalities should budget a reserve to take care of equipment which has a definite life. Information on operating is highly desirable in order to properly set up a maintenance reserve.

Manufacturers Speak

Continuing the practice started last year a symposium was scheduled during which nine representatives of equipment manufacturers gave talks on new ideas being offered by their respective companies. The following persons spoke:

H. B. Snyder, Wallace and Tiernan Co. (Programmed Control Sewage Chlorinator); George H. Shay, Johns-Manville (Transite House Connections); H. B. Crane, Infilco, Inc. (Accelo Filter System of Treatment); James Maskell, Flexible Sewer Rod Co. (Sewer Cleaning Equipment); Dave Gallagher, Worthington Pump Co. (Modern Sewage Pumps); William Sivyer, Chain Belt Co. (Grit Conditioning by Recirculation); Frank Roe, Chicago Pump Co. (Aerated Grit Chambers); Lou Klockner, R. B. Carter Co. (Cross Head Guide and Pneumatic Sewage Ejector).

Allegheny County Sanitary Authority Reports

The Allegheny County Sanitary Authority has issued a report on the Proposed Collection and Treatment of Municipal Sewage and Industrial Wastes in that territory. The report concludes that all of the service area sewage should be delivered to a central point on the Ohio River and the treatment is to consist of pre-aeration, primary sedimentation and chlorination. Sludge is to be digested and the gas utilized for generation of power. It is estimated that the direct construction cost of the proposed plan, including overhead and administrative expenses, will be eighty-two million dollars.

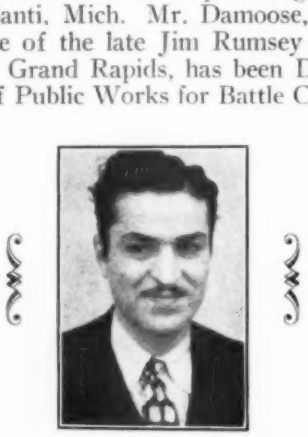
Service contracts with all of the participating municipalities are proposed to extend over the life of the bonds. The report also concludes that legislation needs to be enacted to establish the right of water shut-off for non-payment of sewage bills and to authorize the billing and collection by local water service. Sewer service charges are to provide revenue for operation and retirement of bonds. Rates are to be eighteen cents per thousand gallons per customer up to one hundred thousand gallons, or less, per quarter. For the next one million gallons, or less per quarter, the cost will be twelve cents per thousand gallons and above that, up to two million two hundred and fifty thousand gallons per quarter, the cost will be nine cents per thousand gallons.

It is estimated that seventeen per cent of the householders in the one hundred plus communities in the area will pay no more than fifty cents per month, forty-three per cent will pay no more than seventy-five cents per month, sixty-five per cent will pay no more than one dollar per month and ninety-eight per cent of the householders will pay no more than two dollars per month.

Many mimeographed copies of the report are available at the present time in limited numbers. Persons desiring a copy of the report should write to J. F. Laboon, Chairman, Allegheny County Sanitary Authority, 502 City-County Building. Pittsburgh 19, Pennsylvania.

N. G. Damoose Now City Mgr. at Ypsilanti, Mich.

N. G. (Ance) Damoose has taken over new duties as City Manager of Ypsilanti, Mich. Mr. Damoose, who is one of the late Jim Rumsey boys from Grand Rapids, has been Director of Public Works for Battle Creek,



Mich., for the past several years, and prior to that was Supt. of Sewage at Battle Creek.

"Ance", as he is familiarly known to all water and sewage works men throughout the state of Michigan, is a dynamic personality with a gifted tongue. One of Damoose's most famous non-technical talks on "What Democracy Means to Me" was printed in *Water Works & Sewerage* in 1942.

C. F. Gurnham Opens Consulting Office

C. Fred Gurnham has opened an office as consulting chemical engineer at 72 Lake St., Hamden 14, Conn.

A graduate in chemical engineering from Yale University, Gurnham holds a Master's Degree in Chemical Engineering from New York University, and Doctor's degree in Engineering Science from New York University.

Dr. Gurnham will offer his services for chemical manufacture and industrial waste treatment and disposal. He is a licensed professional engineer in Connecticut and New York, a member of various chemical societies, and has 16 years' experience in various industries.

Public Works Construction for 1947

In the February issue of *Public Construction*, released by the Federal Works Administration of Washington, is the final summation of Public Works construction during 1947.

According to this tabulation, the expenditure for sewage works construction totaled $195,943,000 in 1947. During the year 1946 the expenditure for sewage works was $140,322,000. In 1947 water supply facilities cost $159,322,000 whereas in 1946 the expenditure for the same type of facilities was $131,953,000.

In summation, it appears that in 1947, $355,265,000 was spent for both water and sewage works as against $272,277,000 for the same items in 1946. At first glance this appears to be an appreciable increase, but considering the increased cost of construction during 1946 and 1947 it is quite probable that much of this apparent increase in the money spent for construction in these fields in 1947 was actually spent for increased costs. Therefore, it is doubtful that the actual facilities produced greatly increased during that period.

Considering the increases in cost during the past several years since the war it is obvious that the amount of money expended for construction of water and sewage works facilities has not by any means produced the amount of facilities that were considered necessary for ordinary growth prior to the war. Thus it would appear that construction continues to lag, and that a backlog of needed construction and expansion continues to pile up.

FINANCING SEWERAGE IMPROVEMENTS

By ROBIE L. MITCHELL, Esq.
Attorney
MITCHELL AND PERSHING
NEW YORK, N.Y.

ONE of the most pressing problems in many urban communities is the disposal of sewage and industrial wastes. The pollution of our rivers, harbors and inland and coastal waters into which raw sewage and industrial wastes flow directly or are being pumped has not only impaired or destroyed their value for fishing and recreational purposes, but has already become a serious menace to the health of the people and this menace is increasing more and more with the rapid increase in our urban population. Some of our cities pump their water, including the water which is used in their homes, from rivers which are polluted upstream by the raw sewage of another city or by chemicals and wastes from some mill or factory. It would take a lot of persuasion to make me believe that any city or private corporation has the right so to pollute the rivers and harbors which God has given to all the people of this country.

What are we going to do about it?

For one thing, we can require private industry to clean up its own mess by treating it before dumping it into a river, or by running it through a city's sewage treatment plant and paying for the service.

The Author

The author of this article, with a background of experience in developing plans for financing sewerage improvements, drew up and later successfully defended the Sewer Revenue Ordinance of the City of Miami before the Florida Supreme Court.

In this important case the favorable decision won by Mr. Mitchell from Florida's Supreme Court not only established the constitutionality of the Sewer Service Charge method of financing sewerage improvements in the state of Florida, but also established a precedent of considerable importance in future cases of like nature in all parts of the country.

The author, an illustrious son of Maine, presented this paper before the Maine Water Utilities Association on February 11th, 1948.

Financing the Big Problem

Our big problem comes in financing the construction of the sewage treatment plant.

In the past the construction of sanitary sewers and sewage disposal systems has been financed for the most part by the issuance of general obligation bonds supported by taxes or by taxes and special assessments. In many cities special assessments for all or a portion of the cost of sanitary sewers have been levied upon the property abutting upon the improvement, whether such property is improved or unimproved. The balance of the cost and the cost of intercepting sewers, force mains, pumping stations and outlets and sewage treatment plants in the cases where they have them, have been borne by the taxpayers at large of the cities. As it is extremely difficult, if not impossible, to sell bonds payable solely from special assessments, the cities would issue their general obligation bonds payable from taxes, the amount of the taxes in each year being reduced by the amount of special assessment collections.

But constitutional limitations and restrictions on the issuance of bonds or the incurring of debt have caused us in recent years to consider the possibility and the feasibility of financing the construction of sewage disposal systems by the issuance of revenue bonds, the interest on which is payable solely from the proceeds of sewer service charges (sometimes referred to as sewer rental) to be imposed on those whose sewage is disposed of or treated by the system.

Legal Questions Involved

I was first called upon to draft legislation for this type of financing 17 years ago. We then had very few court decisions to help us as only a few cities had attempted to impose a charge for their sewers. We have been confronted with these legal questions:

1—What are sewer service charges? Are they related to the taxing power, or are they to be classified as special assessments based on benefits? If the former, must the proposed bonds be authorized at an election or must they be included in computing constitutional or statutory debt limits? If they are to be treated as special assessments, should not provision be made for a public hearing after proper notice? Or are they to be treated merely as charges for services rendered?

2—On what factors should the sewer charges be based?

3—How shall the collection of delinquent charges be enforced?

In 1945 I drafted legislative acts to authorize several cities to construct sanitary sewers and sewage disposal systems, to impose sewer service charges and to finance such construction by the issuance of sewer revenue bonds payable solely from such sewer service charges. For fear that the courts might possibly hold that the sewer service charges to be imposed under these special acts should be treated as special assessments, I provided in the acts for a public hearing on these service charges in substantially the same manner as in the case of special assessments.

I then prepared the ordinance for sewer revenue bonds in two of these cities. The conditions in the two cities are not the same. In one about ⅓ of the area, in which about ½ of the population live, had sewers, through which raw sewage flowed or was pumped into a bay in the city's front yard. These sewers had been paid for by general obligation bonds of the city, payable from taxes on all taxable property in the city and additionally secured by the proceeds of special assessments levied on the property abutting on the sewers. The new project involved the laying of additional sewers in the remaining populated sections of the city and the construction of a sewage disposal plant to treat and dispose of the sewage from the entire city. In setting up the financing program it was felt

that it would not be equitable to impose the same charge on property which had already paid special assessments as on the property to be served by the new sewers. So provision was made for companion bond issues, one to cover the cost of the new sewer lines (collection system) and the other to cover the cost of the sewage disposal system, including the *intercepting* sewers, force mains, pumping stations, the treatment plant, etc. The first issue consisted of general obligation bonds payable from unlimited ad valorem taxes and additionally secured by special assessments (for about 60% of the cost) on abutting property. This would place the property owners getting the new sewers on substantially the same basis as those who already had sewers and who had paid special assessments for a part of the cost of their sewers. The second issue consisted of unsecured sewer revenue bonds, payable solely from sewer service charges based on water consumption by the sewer user.

This plan of financing would place some burden on the owners of unimproved property in the new area, special assessments for the new sewer lines in front of such property and a tax to pay a part of the first issue of bonds. The second issue of bonds would be payable from equitable sewer service charges based on extent of use of the sewer in both the old and the new areas.

No One Plan Will Fit Every Case

The consulting engineers suggested that some charge be imposed on all unimproved property in the city in the nature of a "readiness to serve" charge on account of the fact that the construction of the system could be shown of benefit to all property in the city, whether improved or unimproved. But from the decisions of the courts up to that time, including the decisions of the Supreme Court of Pennsylvania in two Philadelphia cases, I feared that any such "readiness to serve" charge would be treated as an exercise of the taxing power, requiring an election on the issuance of the proposed bonds and also the inclusion of such bonds in computing the charter debt limit of the city.

In another city we now propose to include in the sewer revenue financing the cost of the sewer lines in the presently unsewered area and impose an "adjusted" service charge on the people in that area instead of a special assessment. It would appear from two decisions of the Supreme Court of that state that, if the proceeds of special assessments should be pledged to the bonds, it would be necessary to hold an election on the question of issuing the bonds. In another city, where the present sewers are entirely inadequate, many of them must be reconstructed and new sewers as well as a sewage disposal system constructed, the authorizing ordinance which was adopted last month makes a uniform sewer service charge on all users because of the extensive reconstruction required to place the existing sewers in good condition.

I have mentioned these different situations to show that there is *no one plan of financing which will fit every case*. The financing plan for each situation may have to be a tailor-made job and the working out of the setting up of each plan will involve a considerable amount of work and the closest cooperation on the part of the governing authority and its consulting engineers, bankers and lawyers.

Referring now to the three legal questions which have confronted us:

1—What are sewer service charges?

2—On what factors should the sewer service charges be based?

3—How shall the collection of delinquent charges be enforced?

What Are Sewer Service Charges?

In my opinion these questions are closely related and all of them have been convincingly answered by the Supreme Court of Florida in its opinion affirming the decree of the lower court validating a proposed issue of Sewer Revenue Bonds of the City of Miami, reported in 157 Fla. 726, 27 So. (2d) 118, and printed in full in *The Daily Bond Buyer* of August 5, 1946. I feel that this opinion is one of the greatest contributions in the field of municipal finance of any of the court decisions in recent years and that it will be relied on by the courts in other states as the leading decision on the subject.

Prior to preparing the brief in the Miami case, I read every decision on the subject which I could find and a great many articles, some prepared by leading engineers. Analyzing all this and drawing on my very limited knowledge of irrigation law in Montana, where I was engaged in general practice for a few years before getting into the municipal work in the Attorney General's office, I reached the conclusion that, fundamentally, sewer service charges are identical with water rates, that in reality they should be considered to be the second half of the water bill. In paying the water bill we do not pay for the water *itself* as we do for a ton of coal (or a bushel of potatoes) but for its *use*. As stated in my Florida brief,

> "In a modern city a waterworks system would be utterly useless without a system for the disposal of sewage. The service rendered by a waterworks system is only half of the complete service required. Through the waterworks system water which has been purified in accordance with standards prescribed by the proper health authorities is brought to the premises of the consumer for use. Through the sewer system the identical water, carrying the pollution resulting from its use in bathtubs, showers, sinks, washtubs and toilets, is taken from the premises and disposed of in accordance with standards prescribed by the same health authorities. For the service rendered by the waterworks system water rates or charges are imposed. For the service rendered by the sewer system sewer service rates or charges are imposed. The consumer does not pay for the water in either case but for its use, for the service rendered."

I relied to a great extent on recent decisions in New Hampshire, Pennsylvania and Arkansas. The Supreme Court of New Hampshire in Opinion of Justices (1944) 93 N. H. 478, 39 Atl. (2d) 765, had said "sewer rents imposed by the city of Concord are neither taxes nor assessments for a local benefit but, like water rates * * * are charges made for a service rendered." It was contended by the Attorney General that the principle of law which exempts the property of the State from taxation under general statutory provisions also precludes the imposition of a special assessment for improvements upon such property, but it was held that the charges of the city for sewer service are in no sense special assessments.

The City of Philadelphia was authorized by statute to charge and collect "annual rentals for the use of sewers, sewage systems and sewage treatment works," and such charges "shall be apportioned equitably among the properties served by the said sewer, sewage system, or sewage treatment works." In two decisions of the Court it was held that the ordinances passed by the City of Philadelphia did not comply with the statute because they imposed taxes and not charges for the use of the sewers. In the first case (In re Petition of the City of Philadelphia (1940) 340 Pa. 17, 16 Atl. (2d) 32) the ordinance was held invalid because the charge

was made applicable not only to properties actually connected with and accommodated by the sewer system but also to vacant lots and properties not connected with the sewer system. In the second case (In re City of Philadelphia (1941) 343 Pa. 47, 21 Atl. (2d) 876) the rate imposed by the ordinance was a combination of two factors: (1) three mills upon each dollar of assessed valuation, and (2) 25% of the water service charge, or water rent. The sewer rental rate based on assessed valuation of properties actually using the sewer system was held invalid as a tax measured by the value of properties served without relation to the use of the sewers.

In the third ordinance, which was sustained in Gericke v. City of Philadelphia (1945) 353 Pa. 60, 44 Atl. (2d) 233, the charge was "based upon the water consumption of the property served as measured by the charges for water supplied (water service) for the then current calendar year."

Only a short time before filing the brief, the Supreme Court of Arkansas in City of Harrison v. Braswell (1946) 209 Ark. 1094, 194 S.W. (2d) 12, sustained an issue of water and sewer revenue bonds of the City, payable solely from revenues of the municipal waterworks and sewer systems, saying:

> "A sanitary sewer system without water would be a total loss, and a water system without a sanitary sewer system would be impractical, if not entirely useless. So each complements or supplements the other."

In its decision in the Miami case the Florida Supreme Court held that the imposition of fees for the use of the sewage disposal system is not an exercise of the taxing power or the levy of a special assessment, saying:

> "A sewer system is complementary to a water system. A sewer system would be of no value without a water system and a water system would be entirely incomplete without a sewer system. So the principles of law which would apply to one system must likewise apply to the other."

Basis of Charges

Referring to the second question, the sewer service charges in the Miami ordinance were a certain percentage of the water bill, which in turn is based upon the quantity of water consumed as determined by meter measurement. This method appears to be the most equitable and was sustained by the Florida Supreme Court.

Collection of Delinquent Charges

As authorized by a provision we incorporated in the special legislative act, the Miami ordinance provided that the amount of the sewer service charges shall be actually included in the water bill, but each bill showing separately the amount of the sewer service charges and the amount of the bill for water, and that if the sewer service charges are not paid within thirty days from the rendition of the bill, the City shall discontinue furnishing water to the premises and shall disconnect the same from the waterworks system of the City.

In approving this method for the collection of delinquent sewer service charges, the Florida Supreme Court said:

> "Question Three is closely related to Question Two because it challenges the method provided for the enforcement of the charges for the use of the sewage disposal system * * *.
>
> "It is uniformly held that the City may cut off the water being furnished by the City to a consumer when such consumer fails to pay the water bill. * * *
>
> "It appears to us that if no constitutional rights of the owner or occupant of premises are violated by shutting off the water for non-payment of the water bill, no such right will be violated by shutting off the water for non-payment of the bill for use of the sewage disposal system, the two services being so interlocked that neither can be effective without the other."

In other words it becomes a case of "New and Used Water" as one water and sewage works manager has so aptly put it.

Forcing Connections with Sewer System

In cases where the premises of a substantial percentage of the population are using septic tanks and are not connected with the sewer system of the City, we have another problem. Is it lawful to include in the ordinance authorizing sewer revenue bonds a provision requiring the owners of such premises to connect with the sewer system and cease using their septic tanks?

If the construction of the sewage disposal system is to be financed by revenue bonds, payable solely from sewer service charges, a requirement to connect with the sewer system or else pay the charge in any event may be important. But if the construction is to be financed by general obligation bonds or if the statute authorizes, and the constitution of the state permits, the imposition of a readiness to serve charge on such premises, the financing may not be seriously affected.

In sustaining this provision in the Miami ordinance the Florida Supreme Court said:

> "The reasonableness or unreasonableness of the provisions of the section complained of must depend largely, and may depend entirely, upon the character of the terrain of the area affected * * *.
>
> "What may be considered an unreasonable requirement to be applied in a village located on the red clay hills of Leon County might be a regulation vitally necessary for the protection of the public health and welfare of the inhabitants of a city, the larger portion of the terrain of which is not more than 10 or 12 feet above sea level and a less distance above the average water table.
>
> "Private rights must always be subordinated to public rights and the public health is as sacred as any public rights can be. * * *"

Ownership of Water System Important

We have another problem in cases where the ownership and operation of the water system and the sewer system are not the same. Fortunately over 90% of the urban population in this country is served by publicly owned water plants. But where a city or a substantial portion thereof is served by a water plant that is owned by a private corporation, can satisfactory arrangements be made with such corporation for computing and collecting the sewer service charges? Will it shut off the water for non-payment of such charges?

Where the pollution is caused by the sewage and wastes from several cities, towns and villages, we have still another problem. In some cities where I have been working it is expected that the City will construct and operate the sewage disposal system and finance such construction by sewer revenue bonds of the City and then enter into contracts with the adjoining communities for treating their sewage. In some cases where there are several cities and towns, each owning its own sewer system and each being served by a separate water system, it will probably be necessary to enact legislation setting up a sanitary district or a sewer authority covering the entire territory. The prepara-

tion of such legislation will not be an easy task and working out the financing plan on an equitable base will be even more difficult if there are any privately owned water plants within the territory.

The Portland Water District As an Example

You may be anxious to have me make some suggestions in connection with your own situations, particularly here in Portland. But, as I have said, the financing plan for each situation must be a tailor-made job. The Portland Water District is a quasi-municipal corporation created by a special act of the Maine Legislature. The District supplies water to the City of Portland and several adjoining cities and towns. I understand that the City of Portland has its own system of sanitary sewers through which sewage flows untreated into the bays and waters surrounding the City, and that the pollution of these waters is now or may soon become a menace to the health of the inhabitants of the City. If this is true, it appears to me that it would be advisable for the City to have a thorough survey made of the situation, in the first instance by its own engineering, finance and legal departments and then, at the proper time, employ outstanding consultants, particularly consulting engineers who have specialized in this particular line, to cooperate in the preparation of a comprehensive plan for construction of sewage disposal works and the financing of such construction. In the preparation of such plan, consideration must be given to many matters, such as:

1—The type of treatment best suited to the situation, the design and location of the sewage treatment works, intercepting sewers, force mains, pumping stations, and other equipment, the cost of construction and the annual expenses of operation and maintenance.

2—The necessity from a health standpoint of constructing sanitary sewers in any unsewered areas.

3—The extent and the condition of the sewer facilities in the adjoining cities and towns and to what extent the sewage from these facilities contributes to the pollution to be abated.

4—If the construction of a sewage disposal system shall appear to be necessary, by what public instrumentality shall it be constructed and operated, and shall such construction be financed by general obligation bonds or revenue bonds? If all or substantially all of the area to be served is within the limits of the City of Portland, perhaps the City is the proper instrumentality. If so, in determining the type of bonds to be issued, consideration should be given to the margin between the present debt of the City and its legal debt limit and the necessity of preserving that margin for other public improvements.

For a helpful article on "Modern Methods of Sewage Works Financing," which contains a Model Sewer Rental Ordinance, typical Billing Forms used in collecting Sewer Service Charges, and a tabulation of Sewer Rental Experiences, see pages R-161-165 of the 1947 Reference and Data Number of *Water and Sewage Works* (July 1947) and pages R-177 and R-221 of the April 1946 Number.

But regardless of the ability or willingness of the City to issue its full faith and credit bonds, I would urge that careful consideration be given to the matter of issuing sewer revenue bonds, payable from sewer service charges to be imposed only upon those who actually use the system. If this plan should be adopted, however, who will collect the bills? The Portland Water District owns the water plant. Would it like to be a collection agency for the City? If so, how will it enforce collection, and will it be willing to shut off its own water for non-payment of the City's sewer service charges?

Perhaps the Water District is the proper instrumentality. It has done an excellent job in bringing pure water to the premises of those who use it. This, however, is only the first half of a complete service. The other half is the disposal of that water after it has been polluted by the user.

Reasonableness of Charges

Financing the construction of sewage disposal systems by the issuance of sewer revenue bonds, payable from sewer service charges added to the water bill, is comparatively new. There have been some objections to this type of financing on the grounds that the improvement is of general benefit to the community and should be paid for by a general ad valorem tax. I have no legal objection to this plan if the financial structure of the city or town will stand it. But it seems to me that real estate is carrying about all the load that it should at the present time. It is taxed for schools, police and fire protection and many other essential functions of government, both state and local, that cannot be financed in any other way. But in the case of many of our public facilities and utilities I believe that it is more equitable to require the users to pay for them.

What Will the Sewer Service Charges Amount To?

In an ordinance which I have recently prepared authorizing Water and Sewer Revenue Bonds to finance improvements of the City's waterworks system and also additional sanitary sewers and new sewage treatment facilities, the initial sewer service charges to pay the principal and interest of that amount of the bonds which are to be issued for the sewer improvements and to maintain and operate the complete sewer system after the construction of the improvements, will be 65% of the water bill, after making proper adjustment for water not entering the sewer system, such as water used in the sprinkling of lawns, for air conditioning, steam production, etc. In another City for which I am working the sewer service charges will probably be as much as 100% of the water bill.

Are such charges reasonable? Let us consider the service which is being rendered by a waterworks system and the service which will be rendered when the second half of the job is done.

The charge of the Portland Water District for delivering 1,500 cubic feet of pure water to the consumer is $2.50. A cubic foot of water weighs 62.3 pounds, 1,500 cubic feet weigh 93,450 pounds, almost 47 tons. In other words, the water user pays less than 6c a ton for water delivered into his home. What do you have to pay up here for a ton of coal or a ton of fuel oil delivered to your home? For the price of a pack of cigarettes you get three tons of pure water. On the basis of the sewer service charges I have just mentioned, you would pay from 4 to 6 cents a ton for taking away and cleaning up the water after you have fouled it.

It appears to me that such sewer service charges should not be considered unreasonable to the people in any community, particularly if they are fully advised through the press and public meetings how serious a menace to their health the present situation is and how small will be their individual contributions to remove this menace.

The people in a community never object to improving their water system in order to effect cheaper fire insurance. Why not provide some health insurance in addition?

DIGESTION OF SEWAGE SOLIDS

A Review of Established Methods; Comparisons and Comments

IMHOFF TANKS

(The Second Article of a Series)

By LeROY W. Van KLEECK
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WHEN in 1906 Karl Imhoff of Germany introduced the two-story Imhoff or Emscher tank for sewage treatment he made an important contribution to the sewage disposal field. The idea of the two-story tank is said to have originated at the Lawrence Experiment Station of the Massachusetts Department of Public Health in an effort to develop a settling and sludge digestion unit, which would not have the inherent disadvantages of the septic tank. Travis of England also developed such a tank, but he introduced some sewage flow directly into the sludge compartment on the theory that amongst the benefits derived there would be a washing out of excessive enzymes or toxic products of bacterial activity. His idea did not gain favor in America and no Travis tanks of which the writer has knowledge were ever built in the United States.

In contrast, hundreds of Imhoff tanks have been and are being built, many of which continue to give very satisfactory results, while others have given operating difficulties and are being replaced with the plain-settling separate-sludge digestion type of plant. At a time when many post-war sewage treatment plants are being designed it is well to consider the reasons for past operating difficulties, under what local conditions Imhoff tanks should still be built, and how past experience can point to the *tailored design and operation* required to produce satisfactory results.

INFLUENCING FACTORS In Adoption of Imhoff Tanks

The choice between Imhoff tanks or some other method of sludge digestion hinges largely on the general factors for the selection of a method of sludge digestion as outlined in the first article of this series, namely: Plant location, connected population and flow, the character of the sewage, the desirability of utilizing sludge gas,

The Author

the type of operating personnel, and the funds available for construction and maintenance. These factors will now be discussed in order.

Plant Location

Imhoff tanks should be built on relatively isolated sites at least 500 to 1000 feet from houses. Odors may occur due to disturbance of the scum in the gas vents, skimming operations in the flowing-through compartments, and the escape of sludge gas to the atmosphere.

Connected Population and Flow

Although some large cities are still employing Imhoff tanks, the writer is of the opinion that present day conditions militate against their use for the larger communities. Much of this opinion is based on the manual attention required to properly operate Imhoff tanks, and the difficulties today of securing laborers, even at prevailing high wage scales, to do such labor. Many communities are finding it increasingly difficult to adequately man large batteries of Imhoff tanks. Good workmen are in such demand that they are simply not interested in shoveling, skimming, plunging and squeegeeing non-mechanized sewage tanks. Modern design can still provide, however, a highly satisfactory Imhoff tank lay-out for the smaller community where a suitable site is available.

The writer believes a population from 500 to 15,000 persons would include most cases where Imhoff tanks may well fit into the picture. Large institutions provided with available local help, and towns in a population range from 1500 to 5000 offer special situations for the Imhoff tank as an economical method of sewage treatment and sludge digestion.

Character of the Sewage

Imhoff tanks will handle any given sewage as efficiently as plain-settling tanks unless certain harmful industrial wastes are present. Such wastes might likewise preclude the use of separate-sludge digesters with plain-settling, necessitating the drying and disposal of raw sewage solids. The odor from stale or septic sewage can be controlled by pre-chlorination without affecting sludge digestion in the sludge compartment. In fact pre-chlorination will frequently control foaming of Imhoff tanks as well as prevent carrying odors.

Utilization of Sludge Gas

One of the major benefits of separate-sludge digestion is the utilization of the by-product sludge gas. Plants serving a population range from 1500 to 5000, however, produce insufficient gas to provide more than a small portion of the fuel required for heating in northern climates. Hence, gas utilization for this population range is generally of minor consideration. Septic tanks, while they can be operated to produce good results, present inherent difficulties in this population range. The choice between the Imhoff tank or unheated separate-sludge digester frequently hinges on the type of operating personnel and the funds available for maintenance.

Some Imhoff tanks have been designed to collect gas from the gas vents, but the practice is subject to operating difficulties and has not been extensively employed.

Type of Operating Personnel

The difficulties of securing operating personnel for Imhoff tanks in large cities has already been men-

*Conn. State Dept. of Health.

tioned. When personnel is obtainable at reasonable wage scales, the total cost for labor will generally be lower than for any other type of primary sewage treatment plant except septic tanks. No mechanical ability is required unless pumping is involved, and a minimum of technical training is needed. Thus unskilled labor can be employed under the supervision of a foreman or chief operator whose qualifications will be largely dependent on the size of the plant. This factor, coupled with the automatic, nonmechanical, and continuous operation of Imhoff tank treatment, is a major argument in favor of the Imhoff tank.

Funds Available for Construction and Operation

While the construction of Imhoff tanks should usually prove cheaper than for separate-sludge digestion type plants, the reverse may be true. As with any type of treatment the cost per million gallons of sewage treated, or per capita served, will be lower as the size increases. The operating cost of Imhoff tanks, however, for plants of approximately equal connected population should be considerably less than for separate-digestion type plants. Costs of operating plants in the population range of 15,000 to 30,000 in Connecticut average about 38 cents per capita per year. Smaller plant costs may run twice this. Some large cities using Imhoff tanks report yearly operating costs as low as 4 cents per capita. In general, the choice in favor of Imhoff tanks versus another type of plant will not generally be greatly influenced by the funds available for construction, but if low maintenance costs are desired and the other influencing factors are favorable to Imhoff tanks, this type of plant will prove most economical, the per capita cost being frequently one-quarter to one-half that for separate-sludge digestion.

Hand Skimming Operation at an Old Imhoff Tank Plant

DESIGN REQUIREMENTS

Following are pertinent design requirements. The items listed do not include all of the factors which enter into the design of Imhoff tanks but they do represent those considerations which the writer's experience have shown to be particularly important.

1. The *minimum sludge storage capacity,* from hopper bottom to 18 inches below the slots, for tanks in northern climates should be *at least 3.5 cubic feet per capita* served, and preferably 4 to 5 cu. ft. Experience indicates that most of the operating difficulties with foaming, excessive scum accumulations, solids rising through the slot pronouncedly, odors and poor sludge quality would not have been experienced had the tank storage space for sludge been more liberal than originally considered sufficient. If the Imhoff tank is to receive secondary tank humus sludge, the above sludge capacities should be at least 5 to 6 cu. ft. per capita and preferably more. The digestion of activated sludge in Imhoff tanks along with crude solids is not considered practicable and cannot be recommended.

2. The *flowing-through chamber capacity* should provide a detention of *not less than 2½ hours* for the average dry weather flow. Surface area loadings should, furthermore, not exceed *1000 gallons per sq. ft.* (preferably 800) of flow-chamber surface for the 24-hr. average dry weather flow. With reasonable operating attention these design factors will afford a removal of suspended solids equal to that obtained with plain-settling type tanks. Table I gives the analytical results of the examination of Imhoff tank influent and effluent samples collected at a number of plants over a period of 8 years.

3. *Pre-treatment of sewage prior to Imhoff tank* settling should include screening through *1-inch to 1½-inch clear-opening bar screens followed by a grit chamber.* The 1-inch screens for the larger plants should be mechanically cleaned. The 1½ inch screens for the small plants may be hand cleaned. Generally two or more narrow hand-cleaned grit chambers will suffice for the smaller plants. Grit chambers should be provided regardless of the plant's size or whether the collecting sewers are combined or separate. These units are inexpensive and have almost without exception proved their worth at one or another time.

Many Imhoff tanks fail to operate properly because of accumulations of rags, paper and other fibrous materials in the gas vents, and grit in the sludge compartments. Modern design can correct these former faults by giving the Imhoff tank the same break we have given the plain-settling tank. The writer favors screening of sewage for Imhoff tanks with actual removal of the screenings, rather than disintegration of screenings with their return to the sewage flow. Removal of the screenings reduces the tendency for formation of coarse slow-digesting solids in the gas vents which contribute to scum matting and choking of these channels of gas escape.

4. The *fewer the hoppers and dividing walls in the sludge compartments, the better.* Restrictions in the lower compartment of Imhoff tanks tend to segregate sludge deposits, interfere with effective seeding of the raw solids, result in "lime pockets" when lime is added to the digestion compartment, correct acidity, increase the cost for sludge piping and valves, and necessitate more frequent reversal of sewage flow through the tank. Tanks with flat-bottoms and those with continuous or single hoppers have worked very successfully without any flow reversal. Where cross-walls are required for structural strength they should be built astride the flow-chamber sides just above the slots but not higher than 3 feet below the tank flow-line. This will give unobstructed gas vent surfaces for the scum removal lay-out suggested later.

5. *Imhoff tank slots* should be baffled by adaptation of the scheme of

Table I

RESULTS OF ANALYSES OF IMHOFF TANK INFLUENT AND EFFLUENT SAMPLES

Plant No.	Population Served	Samples Collected Over 8 Yr. Period	Sample of	Av. Susp. Solids, ppm.	Av. Settl. Solids, ml./liter, 2 hrs.	Av. B.O.D. ppm.
1	500	23	Raw sewage	200	5.7	370
		23	Tank effluent	90	0.3	250
2	1,850	15	Raw sewage	200	7.2	180
		15	Tank effluent	40	0.4	47
3	10,500	26	Raw sewage	115	5.0	100
		26	Tank effluent	45	0.5	60
4	500	13	Raw sewage	185	6.0	205
		13	Tank effluent	80	0.2	175
5	13,000	26	Raw sewage	200	5.5	105
		26	Tank effluent	90	0.8	80
6*	36,000	33	Raw sewage	215	7.0	250
		30	Tank effluent	115	1.0	195

*Overloaded plant now replaced with plain-settling tanks.

the overlapping concrete apron. Triangular beams used between the converging aprons of the sloping walls are unsatisfactory and serve to accumulate a variety of hanging of stringy materials and coarse solids. These beams also serve as a foot-hold for biological growths. In contrast, an overlapping apron is more or less self-cleansing and is far simpler to keep free of deposits and biological growths through use of the squeegee from one side of the tank.

6. In addition to *minimum slope of 1¼ vertical to 1 horizontal* for the sloping walls of the flow chambers, the use of carefully smoothed concrete for the aprons is highly desirable. The smoother the concrete, the better and easier will be the squeegeeing job. Sludge compartment inspection manholes in the aprons should not be built. They are rarely level with the aprons and may become dislodged.

7. The same care should be given *Imhoff tank inlet and outlet design* as is given the plain settling type of tank. Poor distribution of flow to tanks, clogging of narrow inlet gates, and lack of sufficient effluent weir length are common faults in the older tanks. The use of narrow gate outlets or submerged pipe outlets should particularly be avoided in favor of full-width weir overflows.

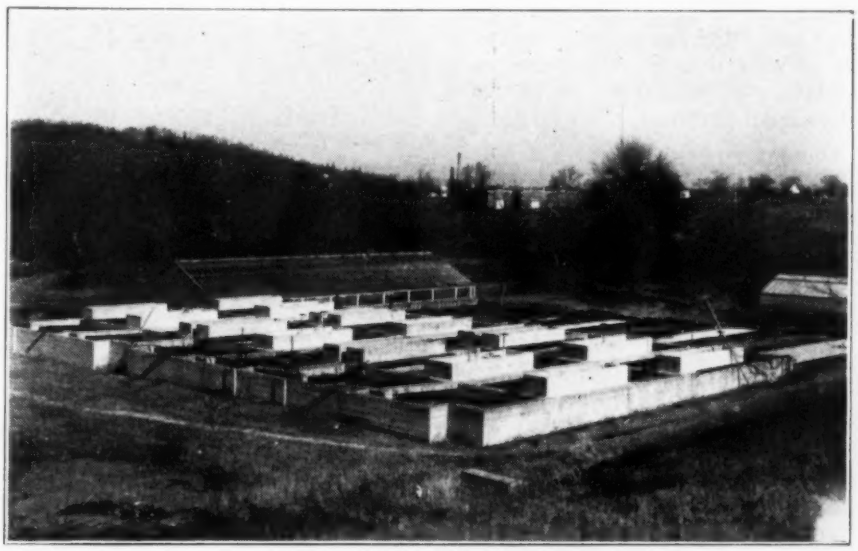
8. *Deep tanks function better* than shallow tanks because of greater sludge storage depths. They afford better opportunities of withdrawing sufficient quantities of ripe sludge without drawing off too much seed material or partially-digested solids. Experience indicates that in general tanks 25 to 30 feet deep will produce better sludge, have less scum and give rise to less foaming difficulties than tanks 20 to 25 feet deep. Tanks less than 20 feet deep are usually unsuccessful in operation.

9. All *valves* should be located outside of the tank for accessibility and again to provide unobstructed gas vent areas.

10. All *walkways* for squeegeeing and skimming operations should have a minimum width of 15 inches. This width will permit reasonably safe tank maintenance without the need of supplemental wooden planks which eventually rot or may become dislodged.

11. In those rare cases where sufficient head is available, a *bottom sludge draw-off*, pitched continuously and uniformly downward from the tank's bottom to the sludge drying bed or lagoon, is highly *desirable*. Such a draw-off will keep accumulations of lime, foreign objects and grit moving out with the bottom sludge. The *second-best* arrangement is a *riser type* draw-off with the horizontal take-off several feet below the flow-chamber slots. This will permit drawing down of the tank level for flow-chamber and slot inspection, a highly desirable arrangement. If a *supernatant liquor draw-off* to a similar depth is installed this will offer the same advantage, and without loss of the seeded digested sludge from the bottom. The increased cost for a supernatant draw-off connection is not great. The initial available head on the sludge outlet line should never be less than 5 feet. Sludge lines should have a minimum size of 8 inches.

12. An all too frequent major operating difficulty with Imhoff tanks is the *scum accumulations in gas vents*, primarily because no facilities have been provided for scum removal except by the time-worn wheelbarrow parade. While gas vent scum draw-offs through pipes or sliding gates are far from new, it is surprising how few Imhoff tanks have been equipped with them. The gas vent area should be unobstructed, and one or more bell-shaped flanges attached to 8-inch or larger pipes should be set in the vents at an elevation 6 inches under the normal flow line. The pipes should be valved outside the tank and connected into the sludge draw-off line discharging to the sludge drying beds or a lagoon. A depth of 6 inches is about right; less depth hinders scum removal and frequently causes scum clogging of the piping, while greater depths withdraw too much subnatant liquor. Sliding gates installed on the outside walls at each end of the gas vents have been used at some plants and should prove satisfactory substitutes for pipe flanges.



Modern Imhoff Tank Plant

Serving 14,000 persons this plant is equipped for skimming draw-off through the sludge line or to drying lagoons.

13. Hand-skimming of Imhoff tank flow-chambers should not be necessary in this day and age. In many plant lay-outs the prevailing wind will concentrate surface skimmings at the removal end of the chambers. A floating scum board, the width of the chamber, can be dragged from one end to the other, or a water spray with a hand hose or permanent water pipe nozzles can be used to drive and con-

centrate the scum. A slotted swivel pipe can then be used to remove the skimmings by gravity to a concentration well or directly to a special drying bed. Such draw-offs are commonly used for rectangular plain-settling type tanks. One or more draw-off pipes fitted with bell flanges and pipe valves just under the tank surface may also be used as a substitute. Such draw-offs and valves help to reduce much of the time consuming labor required for hand skimming and are well worth the small additional cost.

OPERATING REQUIREMENTS

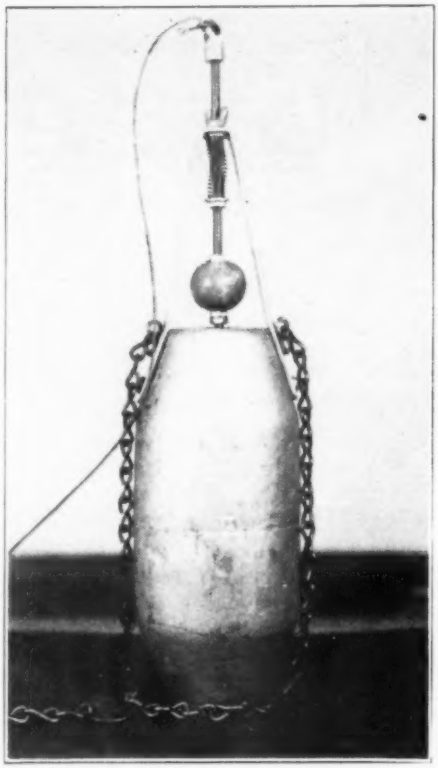
The following requirements are not intended as a complete guide for plant operation, but in the writer's opinion will help to avoid some major operating difficulties. Most operators will agree that operating troubles center around foaming, scum accumulations, carrying odors, and rising sludge in the flowing-through chambers. Poor sludge quality plagues some badly overloaded plants.

Foaming

When frothy material rises in the gas vents above the normal water surface, this is called mild foaming. Real foaming results in the overflowing of froth from the gas vents into the flow-chambers and onto the surrounding ground. A general mess is the result. The writer is grateful that he has not had to cope with a real case of foaming for many years.

Foaming is invariably associated with overloaded plants or new tanks started without seeding in cold weather. In northern climates spring may bring the foam, attributed to be largely soapy materials which rise with the vigorous gas production accompanying accelerated spring activity in accumulated dormant winter sludge.

Deep tanks with adequate storage will foam less than shallow tanks with their limited sludge storage. Foaming may occur regardless of whether alkaline or acid conditions prevail in the digestion compartment liquor and sludge. If an acid condition is present (indicated by a total alkalinity below 1000 ppm. or a pH below 6.6), moderate amounts of lime will sometimes stop foaming.* Excessive hosing of the gas vents actually aggravates foaming, interferes with sedimentation and increases the chlorine demand of the tank effluent. The hydraulic head produced in the digestion compartment as the result of hosing the vents seeks a level through the slots, resulting in displacement of supernatant liquor and frequently of solids into the flowing-through chambers.



Sludge Sounder and Sampler

This commercially available sludge sounder has ball stoppers at top and bottom. Submerged to depth desired, a jerk on the cord opens both ports and the sampler fills. The lowering chain is tagged at one foot intervals to determine depth.

The best cures for foaming are:

1. Rest the tank if possible.
2. If all tanks must be kept in operation, draw some sludge, leaving in plenty of seed sludge.
3. With overloaded tanks, maintain alkaline conditions by small *periodic* additions of milk of lime or dry hydrated lime to the gas vents, working the lime into the scum with a paddle or other manually operated agitator. Avoid dumping large quantities of lime into the vents at one time. This will cause lumping and building up of lime pockets under the slots with eventual difficulties with sludge digestion and scum formation.

Scum Accumulations

Excessive scum accumulation in gas vents is abnormal and indicates either an overloaded plant or lack of operating attention. The writer believes in giving gas vents attention. Scum should be lightly hosed, paddled, limed and otherwise encouraged to settle when weather conditions permit. Holes should be maintained through the scum for gas release. Several times a year rags and other coarse materials in the gas vents should be forked out. Slow digesting or non-digestible material like match sticks and hair should be scooped out of gas vents at least once a year. When a draw-off for the scum is available this, of course, is used instead of the hand scooping.

If valved pipe draw-offs have been installed in the flowing-through chambers for skimmings removal, one method of removing scum from the gas vents is to draw down the tank as near to the slots as possible. Much scum will rise through the slots and accumulate in the flowing-through chambers, whence it can then be floated into the draw-off pipes.

If after a period of years, grit, lime and other heavy solids accumulate in the sludge compartments, normal seeding and digestion will be seriously affected with the likelihood of excessive and persistent scum build-up in the gas vents. Although a major job is frequently in prospect, the tank in such cases may need to be dewatered and completely cleaned. In some cases an ejector lowered to the hopper bottoms can be employed to scavenger the grit and silt laden hoppers. When good water pressure is available, or a portable pump is employed to provide effluent under pressure, a crude ejector can be constructed of pieces of pipe and pipe fittings if a suitable ejector cannot be procured.

Odors

Cleanliness is good insurance against odors. Pre-chlorination (mentioned earlier) will markedly reduce odors from septic sewage. By intelligent operation even very local odors from Imhoff tanks can usually be kept below the nuisance stage.

Rising Sludge

The floating of partially-digested or digested sewage solids in the flowing-through chambers of Imhoff tanks is generally the result of:

1. Sludge depths higher than 18 inches from the slots, generally at or above the slots.
2. Scum depths extending downward to the slot level, so that solids break off from the bottom scum layer and rise through the slot into the flow chamber.
3. Dirty aprons or clogged slots.
4. Excessive hosing of scum.
5. Structural defects in the slot.

*Suggested quantities of hydrated lime may vary from 15 to 80 pounds daily per million gallons of sewage treated, or 2 to 10 pounds daily per thousand people connected.

(Wrong construction, insufficient overlap, broken edges, etc.)

6. Seasonal over-turns caused by temperature changes within the tank.

The remedies for these conditions are either self-evident or have been previously discussed.

Sludge Quality

Sludge quality is mostly dependent on plant capacity; secondly on the method of operation. Draw sludge slowly, in small quantities and frequently! In late fall in cold climates, draw sludge until winter storage requirements are secured. Hold off spring drawing in the north as long as possible. Infrequent drawing of large amounts of sludge is not good practice under any conditions and may lead to much trouble. Table II gives some analyses of Imhoff tank sludges from tanks of adequate capacity. Such sludge is a pleasure to behold, being almost jet black in color, strongly tarry in odor, freely dewatering, and of good weight. The most modern of separate-sludge digesters frequently fail to produce any better product.

Tools

A word about tools. Frequent use should be made of a reliable sludge sounder, such as that pictured in the accompanying cut. Pitcher pumps are awkward and messy in operation by

> In the third article of this series Mr. Van Kleeck will discuss unheated separate sludge digesters and their operation.—Ed.

comparison. Weighted ball-floats or metal discs are not accurate for sludge soundings.

Other tools should include several squeegees with short and long handles (one for the vertical walls and one for the sloping walls and slots), round or square scum plungers (churners) of metal or wood, a scum pusher, if you are blessed with scum draw-offs, a hand skimmer of ¼-inch wire screening or fish netting, if a mechanical skimmer is not available), and a chain slot-cleaner. Squeegeeing of the flow-chamber aprons and cleaning of the slots should generally be a weekly chore.

The tools for Imhoff tanks illustrated in most current books and releases are too heavy and of awkward size. The energy expended in manipulating these tools is too great. Squeegees should be 24 inches wide, and the belting or rubber extended not more than 1¼ inches beyond the holding frame. Skimmers should be 6 inches deep, but not over about 10 x 18 inches in plan. For a drag chain, use one about 3 feet long (not 6 feet) and weighing around 5 lbs. Twisted link chain of 7/16 inch stock is about right.

Conclusions

Imhoff tanks of proper capacity and depth, and with good operation, will function satisfactorily. Given a fairly isolated location they may economically serve communities and institutions in the population range from 500 to 15,000, in particular from 1500 to 5000. Unskilled labor can be used, and per capita operating costs are low.

Such plants should be provided with pre-treatment units providing screening and grit removal, and equipped with mechanical aids for withdrawing gas vent scum and flow-chamber skimmings.

Table II

ANALYSES OF IMHOFF TANK SLUDGES

Sample No.	% Moisture	% Fixed Solids, Dry Basis	% Volatile Solids, Dry Basis	Total Alkalinity, ppm.
1	94	57	43	1300
2	92	51	49	1200
3	94	48	52	1900
4	93	63	37	1100
5	90	49	51	1400
6	80	54	46	2900
7	92	55	45	1700

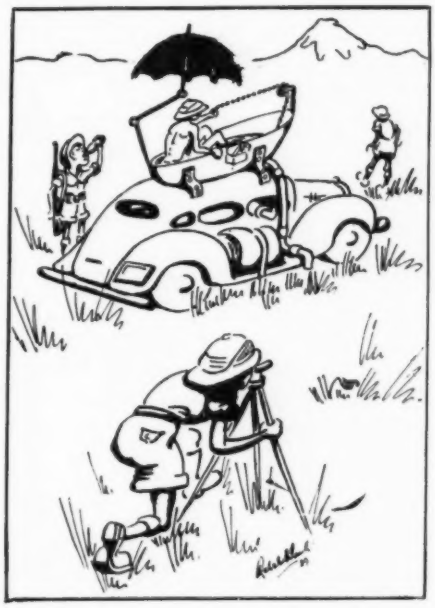
That Vortex Again!

A reader in New Zealand who has been noticing the various items appearing in this magazine from time to time concerning the behavior of the free vortex has sent to us the accompanying humorous sketch. It is captioned — "THE STRASLACKER-EQUATOR EXPEDITION TO DETERMINE WHETHER THE FREE VORTEX REVERSES ITS DIRECTION OF ROTATION WHEN LATITUDE 0° IS CROSSED."

This sketch brings to mind the report by one of our readers who was in the bath tub on shipboard while crossing the Equator. He hurried to inform us that while emptying his bath the whirlpool—over the drain—actually changed direction of rotation while he watched it.

What the chap working on the bottle in the sketch is trying to prove we can't imagine, unless it is that the direction of flow of "free beer" is NOT influenced by latitude.

This sketch originally appeared in *The Fulcrum* published by Witwatersrand University, and was reprinted by George Kent, Ltd. (Lupton, Eng.) in their Information Bul. No. 176. It was sent to us by J. B. Rowntree of the Metropolitan Drainage Board of Auckland, New Zealand.



Ray Derby Heads Los Angeles A.S.C.E.

Ray L. Derby, head of the Sanitary Engineering division of the Los Angeles Dept. of Water & Power, has been elected president of the Los Angeles section of the American Society of Civil Engineers.

A member of A.S.C.E. since 1926, Mr. Derby served as vice-president of the local section last year and as secretary in 1939 and 1940.

Ed. Note: We take this opportunity to correct a typographical error in our editorial of September, 1947, concerning "Vortex Behavior," in which we credited Mr. E. Bruce Hall of Kilmarnock, Scotland, with the explanation of the difference in direction of rotation of free vortices in the Northern and Southern Hemispheres. The gentleman was E. Bruce Ball and not Hall. Thanks to our several readers who pointed out to us this error.

GRAPH FOR DETERMINING SIZE OF SANITARY SEWERS

JOHN G. MONTGOMERY
Civil & Sanitary Engr.
WASHINGTON, D.C.

FORTY to fifty years ago engineers were applying the formulas of Craig, Dredge, Dickens, Kirkwood, McMath and Burkli-Ziegler. None of these formulas was found satisfactory for all cases in the design of storm sewers, because none takes into account all functions and conditions. Also since the authors of such older formulas were European or derived their data from European sources, the maximums were those for Europe and were not applicable to this country. Field results proved this to be true —the maximums were deficient; sometimes by 200 per cent.

The design of sanitary sewers was placed within the same design category as storm sewers, for the reason that most of the workable data at that time were based on some local "plumbers' rule-of-thumb" method—and a greatly exaggerated need of large vents and stacks. I interrogated a plumbing engineer on the question of the size house drains and stubs and the reason for the large factor in design between the actual water demands and the "spill" capacity of the building drains. The engineer's explanation was that he thought the apparent large area in proportion to the water demand was necessary to compensate for maximum demands, *surges* and stoppages within the system.

I have had many years experience "across the board" designing and checking outside collecting systems for sanitary sewers. During this period I have found a great number of installations where the plumbing engineer has stubbed out from the building with pipes many times larger than required for "actual demand." Actual demand in this case included an allowance for "maximum flows, *surges*, and stoppages."

I have included with this article a graphic diagram that will be of some help in determining the proper size of sewer pipes. This graph can be used in determining the increment from all or a single building tributary to the sewer. A check can be made for the proper size of outfall sewers to determine in advance of construction if the

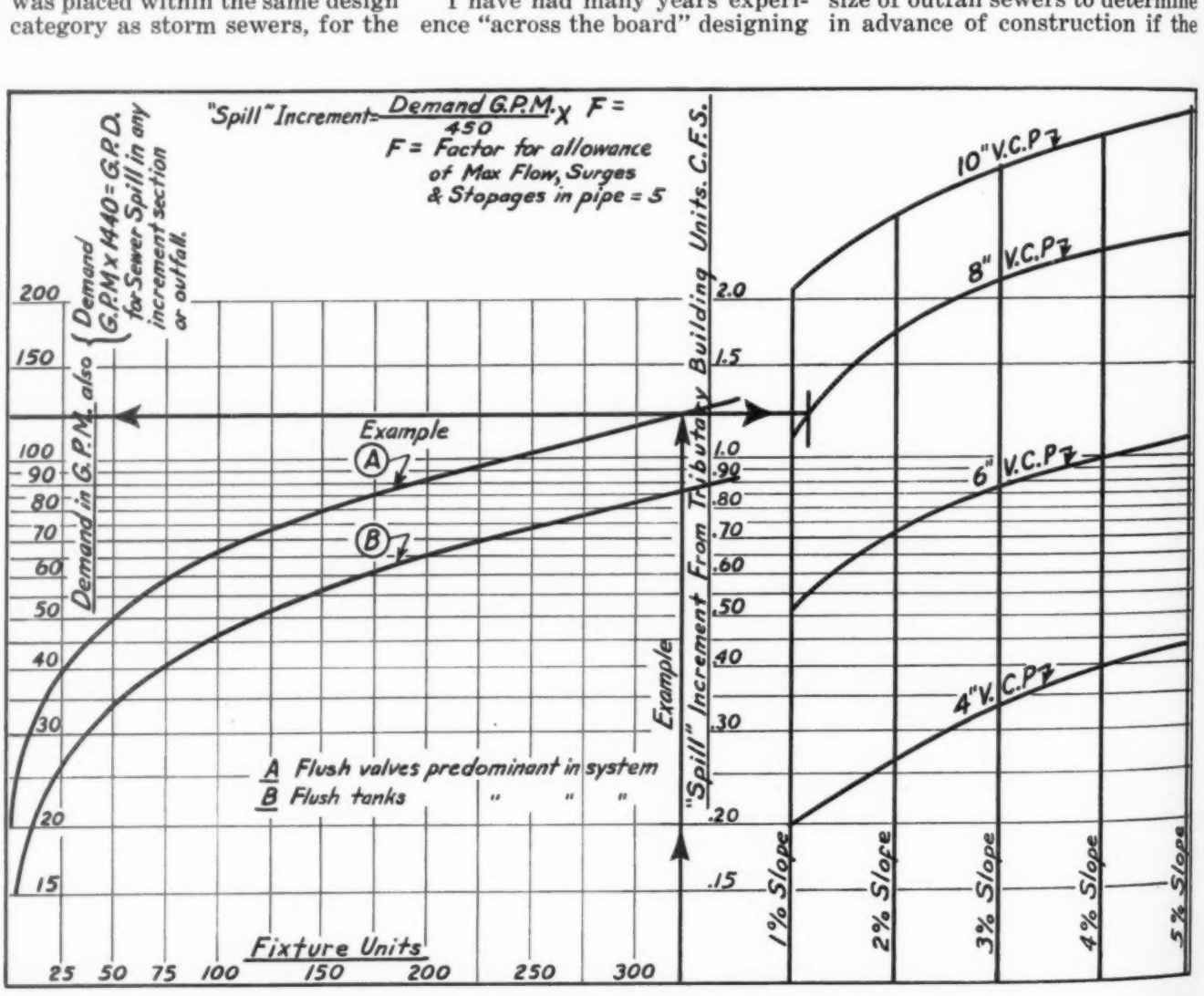


Diagram for Determining the Proper Size for Sanitary Sewers

city sewer will be large enough to take the "spill."

Such a check will be determined from an example as given in the accompanying table, the data of which are taken from an imaginary plot plan.

Referring to the table it is shown that the total flow for this imaginary plot is 2575 gal. per min. Plumbing engineers, in designing the capacity and size of service pipes, express the amount of water to be carried in "Fixture Units." The "fixture unit" is based on the rate of discharge from various types of fixtures and has been set as equal to a discharge of 7.5 gpm. per fixture unit. Therefore if we divide 2575 by 7.5 it is determined that the flow from that plot of buildings is equivalent to 343 fixture units.

The next step is to refer to the diagram. Enter the diagram at 343 fixture units and proceed upwards on the chart until the line intersects with Curve A. Extend a line from that point on Curve A to the right and to the left. Where this line intersects the "Spill Increment," read 1.22 cu. ft. Continuing to the right this line will intersect one of the pipe size Curves. In this example it is indicated that the pipe size needed will be 8-in. and the slope will be just slightly more than 1 per cent. Thus has been determined the required pipe size and slope. If the sanitary facilities were of the flush tank type, Curve B would be used.

TABLE OF DATA FOR SEWERS IN EXAMPLE

No. of Bldgs. in Plot	Size of Service—Inches	Max. Cap. of Service Pipes—gpm.	Max. Flow Possible gpm.
1	¾	7.5	7.5
2	1½	24	48
2	2	40	80
1	2½	55	55
1	3	85	85
4	4	150	600
5	6	340	1700
Total			2575.5

The total expected water consumption in gal. per day can be determined also by extending the horizontal line from the point on Curve A to the left until it terminates at 122 gpm. demand (or 100 times the "Spill Increment"). Obviously this demand does not exist for the full 24 hours, and to obtain the daily flow it is necessary to adjust this demand by the proper capacity (or load) factor which takes into account the number of hours of actual service. This capacity factor is expressed as per cent, being obtained by dividing the service hours by 24. This capacity factor will vary depending on the type of project being served. For example, apartments will have a capacity factor of approximately 50 per cent, hospitals 75 per cent, factories 40 per cent. For the above example, let us assume a capacity factor of 60 per cent. Then the minutes of use during the day will be 864 (24 x 0.6 x 1440). The total daily demand will then be 864 x 122 or 105,410 gal. per day of sewage flow.

It is estimated that the sewage flow is 80 per cent of the water consumption. Therefore the total water consumption in that plot of buildings will be 105,410 x 1.25 or 133,740 gal. per day.

The population to be served by this sewer system can be determined by dividing the total water consumption by 100, thus obtaining 1337 persons.

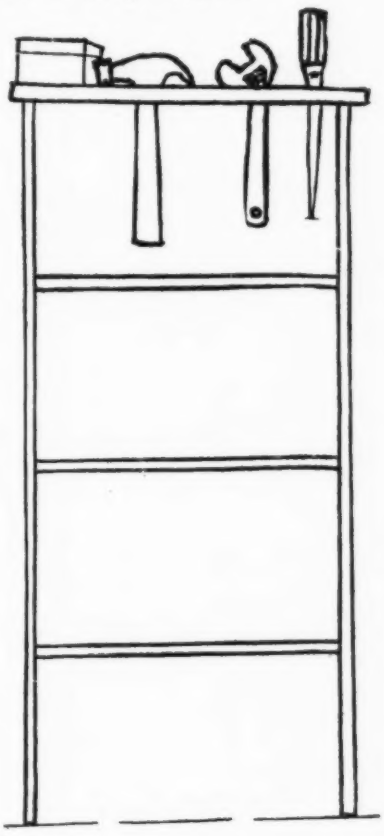
Improve the Value of Your Step Ladder

Operators are continually finding it necessary to use a step ladder in and around a plant. So, since mechanics as well as executives endeavor to be efficient, it is logical that step ladders should be made more efficient if possible.

As shown in the accompanying sketch, a step ladder, or any other ladder for that matter, can be considerably improved by fitting the top with a holding slot or with several holes or devices for safely keeping the tools you use most while at the top of the ladder.

It is a good plan, also, to attach a box to the top containing small brads, nails, screws, etc., also as indicated in the sketch. The box should have a cover that can be fastened securely so that when the ladder is carried to any position the small parts therein will not spill out.

Little improvements like this save much time and energy. It is easy and in the end well worth while to improve a ladder in this manner.



Contributed by W. F. Schaphorst, M. E., Newark, N. J.

Indiana Cities Revolt Against State Orders to Construct Sewage Plants

Soaring construction costs have caused a revolt among several Indiana cities against firm orders of the Indiana State Board of Health, on the advice of the Stream Pollution Committee, to stop delay of construction of sewage disposal systems to correct stream pollution.

The latest city to bypass a Jan. 1, 1948, deadline for start of construction of a sewage disposal system to relieve stream pollution is Decatur. The City Council now is studying the problem. Several years ago the engineering firm of Consoer, Townsend & Associates made a preliminary survey for the city and estimated the cost at that time at $800,000. Today the estimated cost is $1,500,000, almost double.

Under the circumstances, the city is delaying action on the ground that the excessive cost of the project would place an undue burden on the taxpayers of the city.—*Don Wimmer.*

Comments of the Editors

The Green Light (and Some Yellow) on Adopting Sewer Service Charges

In this issue is an important contribution to the literature on "Financing Sewerage Improvements". We urge a careful reading of this article by Robie L. Mitchell, Esq., by water works men as well as sewage works men.

Of particular interest to water works men (perhaps no less so to all readers) is the fact that the courts of the land have come to base their decisions regarding the legality of sewer rental or sewer service charges on the inescapable opinion that sewer service and sewage disposal is *fundamentally a part of the water service.* The principal argument hinges on the fact that the supplying of water to a property entails the provision of sewer service also; and that, in truth, the charge for water service is not a charge for the water as a commodity, but rather is basically a charge for the *use* of the water which in due course of things must also be removed from the premises in the form of *"used water"*, known more familiarly as sewage. Water rent, or water service charges, being in fact a charge for the service which the water performs for the taker, the sewer rental or sewer service charge is merely a complementary charge for the transportation of the "used water" away from the premises and to a treatment plant wherein it is put into condition for safe disposal without danger to the community or the neighbors on the same watershed. In other words, without water service there would be no sewage problem, and without sewer service, water service would be impracticable if not an actual menace to the public health.

In the matter of establishing the validity of the sewer service charge as a means of financing the maintenance and operation of sewerage systems and sewage treatment plants the Miami case ranks high in importance. In his presentation of the claims and his argument of this case before the Florida Supreme Court, Mr. Mitchell benefited from a background of valuable knowledge of irrigation law and experience in developing sewerage financing plans. Needless to say he had also fine-tooth combed all court decisions which pertained to sewerage financing. In having won the favorable decision for Miami the entire sewerage interests of the country should feel at least indirectly indebted to Counselor Mitchell who, incidentally, does not hesitate to give high credit to the engineer when he states that in *any* plan of sewerage financing the soundness of the engineer's estimates of costs of structures, depreciation rates, and costs of maintaining and operating the system, constitutes the all important foundation structure. Such foundation *must* be dependable or else the best possible financing plan will not stand the test of time and experience.

In his briefs in the Miami case Mr. Mitchell argued in part as follows. More importantly, the Florida Supreme Court agreed that *fundamentally* sewer service charges are identical with water rates. In reality the sewer service charge should be considered as nothing more nor less than that portion of the water bill which is necessitated whenever the municipality or corporation is called upon to receive and dispose of the water, after it has served the purposes of the taker, or the "renter". Thus it becomes apparent that in paying the water bill we do not pay for the water *itself*, as we do for a ton of coal or a bushel of potatoes, but rather for the *use* of the water. We actually consume but a small fraction of the water. Thus in paying the water bill, as in paying the sewer service charge, the user is paying for *service* rendered in each instance.

In its decision the Florida Supreme Court had the following to say:

> "A sewer system is complementary to a water system. A sewer system would be of no value without a water system, and a water system would be entirely incomplete without a sewer system. So the principles of law which would apply to one system must likewise apply to the other."

Legality of Collection and Connection Enforcement

Of high importance to the standing of Sewer Revenue Bonds in the finance field is the question of the sewerage authority being able to enforce the collection of the sewer service charge. The most positive method of enforcing collection is the discontinuance of water service for nonpayment of the sewer service charge. Regarding the legality of the water cut off process 30 days after billing in case the sewer charge is not paid in such period, the Florida Supreme Court in approving this practice said:

> "It appears to us that if no constitutional rights of the owner of the property or occupant of the premises are violated by shutting off the water for non-payment for water service, no such right will be violated by shutting off the water for non-payment of the bill for the use of the sewage disposal system, *the two services being so interlocked that neither can be effective without the other.*"

Regarding requirements that the owners or occupants of properties shall connect to the public sewer system when the sewers are available, or when extended to the property in line, the Florida Court supports such requirement on the grounds that—

> "Private rights must always be subordinated to public rights, and public health is as sacred as any public right can be."

Some Unsolved Problems

Mr. Mitchell points out some of the most prevalent problems in the setting up of a sound and workable plan for self-sustaining sewerage financing.

One of these unsolved problems is that of the community served by a privately owned water utility. The question is, will the private corporation cooperate to the extent of computing and billing its customers for the sewer service charge on a metered water basis, or any other basis? If not, will the company provide metered consumption figures and flat rate charges to the sewerage authority for its use in rendering sewer service bills? Also (and of greatest importance) will the water company be willing to shut off water service to a property if the sewer service charge is not paid within a reasonable time after billing?

Mr. Mitchell also calls attention to several other examples of types of problems which must be solved in developing sound sewer service charge plans, and states that experience has shown that no blanket pattern can be developed to fit all situations. In most instances a plan with important basic features can be used as the starting point, but almost invariably some individual "tailoring" of the general plan will then be required if the plan is to become a *complete* plan which is legally and constitutionally sound, and in which financing agencies must be able to assume whole-hearted faith before placing their money in the venture, or before recommending the revenue type of bonds to their customers. It is on this score that the best available financial engineering *and* legal advice and guidance at any reasonable cost should be secured by the municipality wishing to develop a sound pay-as-you-use method of financing sewerage betterments.—L.H.E.

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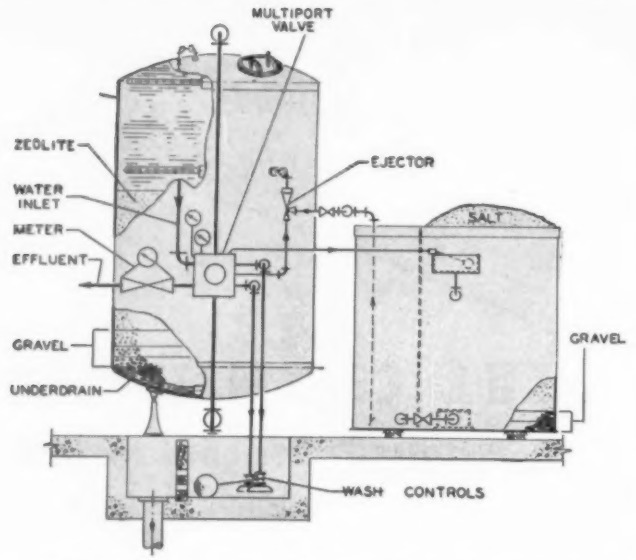

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MEETINGS SCHEDULED

APR. 15-16—BALTIMORE, MD. (Lord Baltimore Hotel)
Maryland-Delaware Water & Sewerage Association. Sec'y-Treas., E. Virginia Gipe, 2411 N. Charles St., Baltimore, Md.

APR. 15-16—CHICAGO, ILL. (Congress Hotel)
Illinois Section A.W.W.A. Sec'y, Carl N. Brown, U. S. Pipe & Foundry Co., 122 S. Michigan Ave., Chicago 3, Ill.

APR. 16—PATERSON, N. J. (Y.M.C.A.)
North Jersey Water Conference. (Final Dinner Meeting) Sec'y, E. Walasyk, 299 Park St., Hackensack, N.J.

APR. 21-23—WEST LAFAYETTE, IND. (Purdue Union Building)
Indiana Section A.W.W.A. Sec'y, Charles H. Bechert, 140 N. Senate Ave., Indianapolis, Ind.

APR. 22—LINCOLN, NEB. (Hotel not selected)
Nebraska Section A.W.W.A. Sec'y-Treas., C. L. Fisher, Deputy City Engr., City Hall, Lincoln, Neb.

MAY 3-7—ATLANTIC CITY, N. J. (Convention Hall)
American Water Works Association (Annual Meeting). Hotels—Ambassador, Chelsea and Ritz Carlton. Exec.-Sec'y, Harry E. Jordan, 500 Fifth Ave., New York, N.Y.

MAY 12—MARLBOROUGH, MASS. (Marlborough Country Club)
New England Sewage Works Association. Sec'y-Treas., Walter E. Merrill, 511A State House, Boston, Mass.

MAY 12-15—BOISE, IDAHO (Hotel not selected)
Pacific Northwest Section, A.W.W.A., Sec'y, Prof. Fred Merryfield, Oregon State College, Corvallis, Ore.

MAY 20—DOVER, NEW HAMPSHIRE
New England Water Works Association, Sec'y, Joseph C. Knox, Statler Building, Boston, Mass.

JUNE 4-5—NIAGARA FALLS, N.Y. (Hotel Niagara)
New York State Sewage Works Association. Sec'y, A. F. Dappert, State Dept. of Health, Albany 1, N.Y.

JUNE 10—SWAMPSCOTT, MASS. (New Ocean House)
New England Water Works Association (Annual Outing) Sec'y, Joseph C. Knox, Statler Building, Boston, Mass.

JUNE 7-11—CHAPEL HILL, N.C. (University of North Carolina)
North Carolina Waterworks Operators' Association. Sec'y-Treas., Max D. Saunders, P. O. Box 1126, 23 Oakwood Drive, Chapel Hill, N. C.

JUNE 7-12—GAINESVILLE, FLA. (University of Florida)
Florida Water & Sewerage Conference and Short Course. G. M. Turner, Ass't Dean, Univ. of Florida, Siegel Bldg., Gainesville, Fla.

JUNE 9-11—AUBURN, ALABAMA (University of Alabama)
Alabama Water & Sewage Association, Sec'y, Alex. O. Taylor, Alabama Polytechnic Institute, Auburn, Ala.

JUNE 9-11—AUBURN, ALABAMA (University of Alabama)
Alabama Water & Sewage Operators Short Course & Conference. Director, Alex. O. Taylor, Alabama Polytechnic Institute, Auburn, Ala.

JUNE 18-19—MILWAUKEE, WISC. (Schroeder Hotel)
Central States Sewage Works Association. Sec'y-Treas., Paul W. Reed, 1098 W. Michigan St., Indianapolis, Ind.

JUNE 23-25—DAYTON, OHIO (Dayton-Biltmore Hotel)
Ohio Conference on Sewage Treatment. Sec'y-Treas., G. A. Hall, State Dept. Health, State Dept. Health Bldg., Columbus 15, Ohio.

MEETINGS SCHEDULED

AUG. 23-25—STATE COLLEGE, PA. (Nittany Lion Inn)
Pennsylvania Water Works Operators' Association. Sec'y-Treas., I. M. Glace, 1001 Front St., Harrisburg, Pa.

AUG. 25-27—STATE COLLEGE, PA. (Nittany Lion Inn)
Pennsylvania Sewage & Industrial Wastes Association. Sec'y-Treas., Bernard S. Bush, Kirby Health Center, Wilkes-Barre, Pa.

SEPT. 1-2—WINNIPEG, CAN. (Royal Alexandria Hotel)
Minnesota Section A.W.W.A. Sec'y, R. M. Finch, 416 Flour Exchange, Minneapolis, Minn.

SEPT. Date not set—MINOT, NO. DAK.
North Dakota Water & Sewage Works Conference. (20th Annual Convention). Sec'y-Treas., Jerome H. Svore, State Health Dept., Bismarck, N. Dak.

SEPT. 14-17—NEW YORK, N. Y. (Hotel Pennsylvania)
New England Water Works Association. Sec'y, Joseph C. Knox, 609 Statler Bldg., Boston, Mass.

SEPT. 14-15—DES MOINES, IOWA (Hotel Kirkwood)
Iowa Sewage Works Association. Sec'y-Treas., Leo Holtkamp, P. O. Box 310, Webster City, Iowa.

SEPT. 22-24—FLINT, MICH. (Hotel Durant)
Michigan Section A.W.W.A. Sec'y, Raymond J. Faust, State Dept. of Health, Lansing, Mich.

SEPT. 29-30—CLARKSBURG, W.VA. (Stonewall Jackson Hotel)
West Virginia Section A.W.W.A. Sec'y, John B. Harrington, State Dept. of Health, Charleston, W. Va.

SEPT. 30—OCT. 1—SIOUX FALLS, SO. DAK. (Carpenter Hotel)
South Dakota Water & Sewage Works Conference. Sec'y, W. W. Towne, State Board of Health, Pierre, So. Dakota.

OCT. 7-8—MANSFIELD, OHIO (Mansfield-Leland Hotel)
Ohio Section A.W.W.A. Sec'y, F. P. Fischer, 812 Perry Payne Bldg., Cleveland 3, Ohio.

OCT. 10-13—GALVESTON, TEXAS (Buccaneer Hotel)
Southwest Section A.W.W.A. Sec'y-Treas., L. A. Jackson, Robinson Memorial Auditorium, Little Rock, Ark.

OCT. 13-15—ATLANTIC CITY, N. J. (Haddon Hall)
Pennsylvania Water Works Association (52nd Annual Meeting). Sec'y, E. R. Hannum, P. O. Box 315, Windber, Pa.

OCT. 13-15—BOSTON, MASS. (Statler Hotel)
American Society of Civil Engineers. (Fall Meeting). Ass't. Sec'y, Allen Wagner, 33 West 39th Street, New York 18, N. Y.

OCT. 13-15—MONTGOMERY, ALA. (Whitley Hotel)
Alabama-Mississippi Section, A.W.W.A. Sec'y, John L. Snow, First National Bank Bldg., Montgomery, Ala.

OCT. 15-16—PHOENIX, ARIZ.
Arizona Water & Sewage Works Association. Sec'y-Treas., George W. Marx, San. Eng. Div., State Dept. of Health, Capitol Bldg., Phoenix, Ariz.

OCT. 18-21—DETROIT, MICH. (Statler Hotel)
Michigan Sewage Works Association. Sec'y-Treas., R. J. Smith, 545 Elizabeth St., E. Lansing, Mich.

OCT. 18-21—DETROIT, MICH. (Hotel Statler)
Federation of Sewage Works Associations. Sec'y-Treas., W. H. Wisely, 325 Illinois Bldg., Champaign, Ill.

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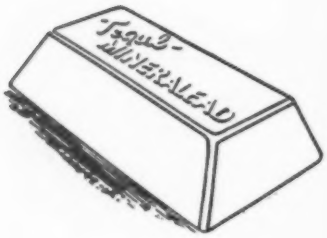

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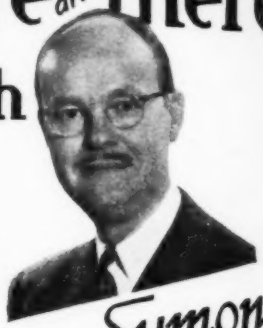
See us at Booth No. 26 at Atlantic City

Here and There with Doc Symons



H.T.M.A.—And need I remind you that *this* is the Annual Reference and Data Issue of *Water & Sewage Works?*—**And** need I remind you further that this R & D issue contains a consolidated index covering all of the articles published in the Reference and Data issues from 1941 on?—AND need I remind you further that in order to have all of the current reference and data material, you need this issue, last year's issue and the 1946 issue as well?—Need I remind you?—Mmmmmm?

* * *

Shades of Bureaucracy—I wish I could remember who told me this story — I think I heard it down Atlanta, Ga., way, when I visited Van Porter Enloe of the Atlanta Sewage Works, "Bill" Weir of the Ga. Dept. of Health, Paul Weir of Atlanta's Water Works, and George Reid and Bob Ingols of Georgia Tech.—

At any rate, so the story goes —a water works reservoir in a highly important war boom town was infested with sea gulls, much to the dismay of the local operator who wrote the Washington big wigs for advice on how to get rid of the gulls.

Back came the answer, "Don't worry about the gulls. They don't drink much water."

* * *

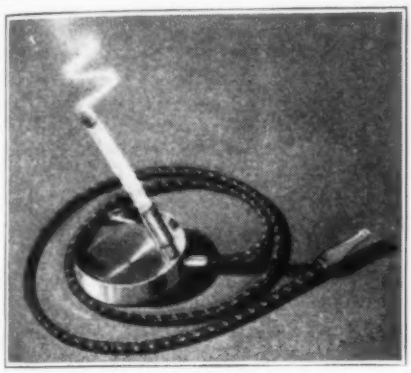
Memo to the Pee Bees! By dint of much persuasion, I was able to obtain a copy of that famous book, "What I Know About Bridge and How I Play It", written by Marsden C. Smith, The Water Man from Richmond, Va.

Tell me, Most Honorable Pee Bees —If I diligently study that book, will I be eligible for membership in your august organization?

Society of Council Bluffers. Not all of the members of the Society weathered the winter too well. Mary VanAtta was hospitalized for a while and still feels kinda rocky, Wendell LaDue had a severe case of the Ohio River Collywobbles, and "yours truly" had to wear a two-way stretch for a while to hold in place a sacroiliac that wandered out of place during a snow shoveling episode.

* * *

Smokers Aboy! Some time back this column carried a cartoon showing the precautions to take if you smoke in bed. On seeing it, one Charles H. Capen, Chief Engineer, of the No. Jersey District Water Supply Commission at Wanaque, N. J. (and Chairman of the A.W.W.A. program committee), sent the picture of the device shown below. He knows where you can get one—for only $3.00.

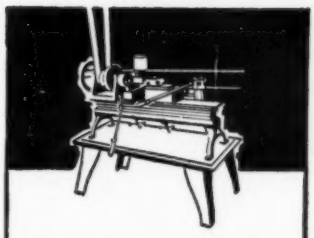


* * *

It Won't Happen Again, Fellows. —When Wilford C. Hagaman, Engineer of Germantown, Philadelphia, Pa., sent in his subscription renewal, he wrote, "I did not renew my subscription before, because I thought the price too high for a small town paper, —small town, because every issue contains numerous pictures of the visiting firemen. I subscribed to your magazine for the technical information, not to look at strips and strips of pictures of your associate editor, — & —'s and other salesmen."

A few days after that letter, John R. Downes, Supt. of Sewage Treatment at Plainfield, N. J. said to me, "Doc, I counted your picture five times in a recent issue, and that's too many." You are absolutely right, John, even the four times it actually was, was about that many, too many.

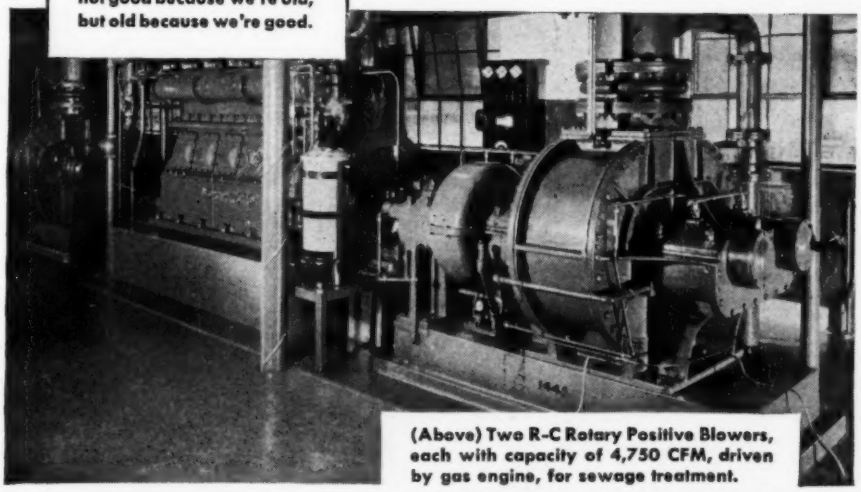
I'm not sure every one agrees with Mr. Hagaman about others' pictures, but as for me—except for the cut at the top of this column, you won't see me again this year unless I die, am nominated for president, or change jobs;—that's a promise—and will you accept my apologies, boys?



The first successful turret lathe and the first Roots Blower were built in the same year . . . 1854. We're not good because we're old, but old because we're good.

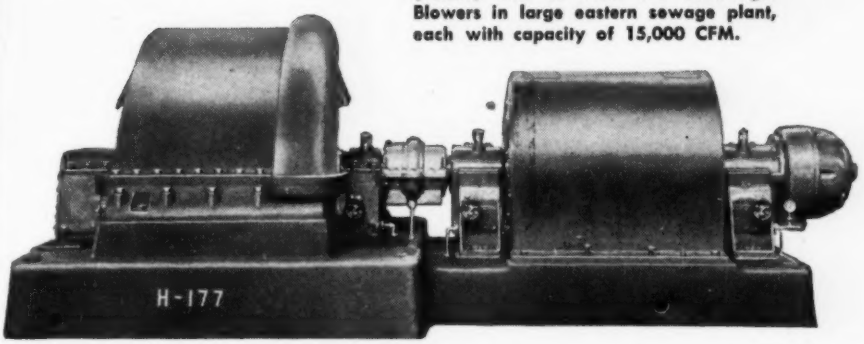
Exclusive Service...

R-C dual-ability



(Above) Two R-C Rotary Positive Blowers, each with capacity of 4,750 CFM, driven by gas engine, for sewage treatment.

(Below) One of three R-C Centrifugal Blowers in large eastern sewage plant, each with capacity of 15,000 CFM.



You obtain an important, exclusive advantage when you consult us about problems of moving air or gas. That is, our *dual-ability* to supply either Centrifugal or Rotary Positive units.

This *dual choice* allows you to match equipment exactly to your needs, in capacities, pressures and other characteristics. You can have standard Rotary Positive units from 10 CFM up to 50,000 CFM, or Centrifugal units from 2,000 CFM up to 100,000 CFM. You save time, trouble and money by using one supply source.

So, utilize this exclusive *dual-ability* by consulting us on all your blower or gas pump requirements. The economy and dependability of R-C Blowers have been proved for almost a century.

ROOTS-CONNERSVILLE BLOWER CORPORATION
804 Mount Avenue, Connersville, Indiana

ROOTS-CONNERSVILLE
ROTARY CENTRIFUGAL
BLOWER BUILDERS R-C SINCE 1854
BLOWERS · EXHAUSTERS · BOOSTERS · LIQUID AND VACUUM PUMPS · METERS · INERT GAS GENERATORS

* * ONE OF THE DRESSER INDUSTRIES * *

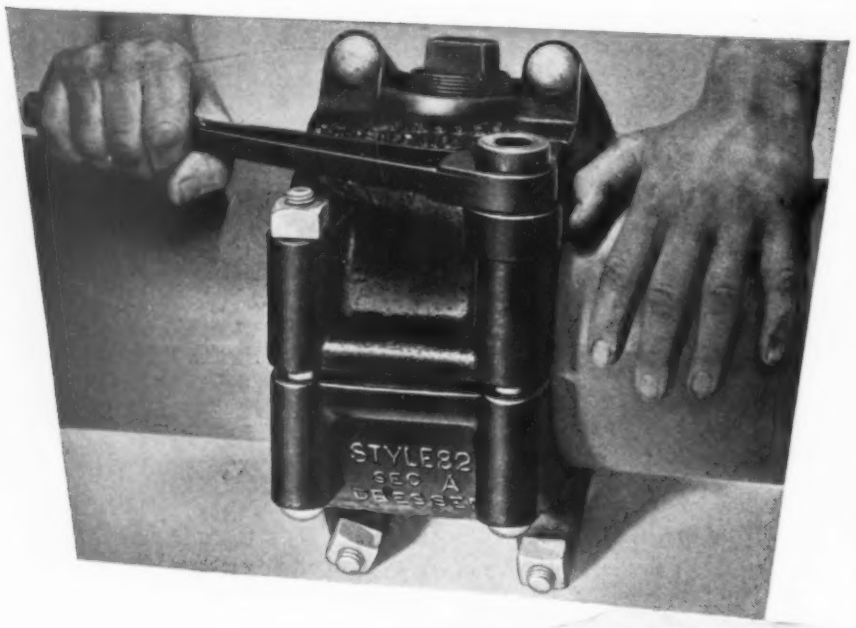
Norma
Mrs. Harry
(P.F.T.) Schlenz
Chicago, Ill.

Helen
Miss Addison
Bank of Amer.
San Fran., Cal.
(Norma's Sister)

"Shorty"
Mrs. Mike
(Neptune Meter)
Siebert
Richmond, Va.

Lillian
Mrs. Daniel J.
(Permutit)
Saunders
New York, N.Y.

"Dot"
Mrs. E. M.
"Casey" (Simplex Valve) Jones
Philadelphia



IT'S AS EASY AS IT LOOKS...

THIS 10-MINUTE WAY TO REPAIR BROKEN LINES

A wrench is the only tool you need to make a *permanent* repair on broken CIP, with the Dresser Style 82 "Adjustable" Repair Sleeve. This simple, sturdy sleeve can be installed, bottle-tight, in 5 to 10 minutes, even on off-size pipe. Interchangeable sections give adjustment range from Class "A" through Class "D" cast iron pipe. Sizes 4", 6" and 8". Write for descriptive Form 441.

FOR LARGER SIZES
Dresser Split Sleeve, Style 57C, encloses breaks, holes, splits in CIP up to 12" diam.

DRESSER **REPAIR SLEEVES**
ONE OF THE DRESSER INDUSTRIES
Dresser Mfg. Division, 59 Fisher Ave., Bradford, Pa. • In Texas: 1121 Rothwell St., Houston • In Canada: 60 Front St., W., Toronto • Sales Offices: New York, Chicago, Houston, San Francisco

Now that the 1948 A.W.W.A. Convention approaches, it seems a good time to finish up the pictures I took at the S.F. meeting last year. Naturally, I have saved the best for the last—And these are the ones you have been waiting for, haven't you? These are but a few of the many similar I wish I could have snapped.

* * *

How many of you listen to "Red" Skelton? Then you know Da-a-a-isy June, Clem Kaddiddlehoppe's girl. Well sir, I met an original Daisy June at the San Fran Conventions but it wasn't Red Skelton's. This one ("Purty" too) was the guest of Sue and Jim (Nat. Water Main Cleaning) Brown, and was the wife of C. B. Patten, City Supt., Lindsay, Calif. Some of my readers may have known them when they lived in Michigan City, Ind.

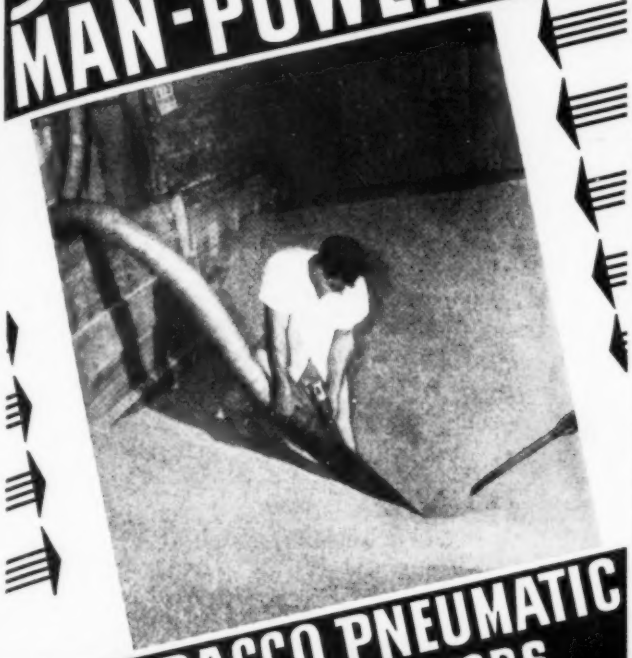
* * *

Some years ago when Linn Enslow covered water works meetings as Editor of *Water & Sewage Works* and not as President of the A.W.W.A., he took a picture of a young Dorrco hopeful, named Allan Darby, learning to do a then popular dance, the Bumps-A-Daisy. The instructor was a comely blonde; yes, ra-a-ther!

Allan, now a respected member of the Pee Bees, was unaware of the camera click. And when the next issue of *W. & S.W.* came out (it was on time in those days) Allan was on the road. When he arrived at one of the cities on his call list, he phoned the super, who had seen that issue.—And when Allan walked into the water works, he was greeted with, "Hands, Knees, and Bumps-a-Daisy," with gestures.

Just goes to show what one of our pictures can do toward furthering the fame of an up and coming young man.

Save . . . MAN-POWER



With DRACCO PNEUMATIC CONVEYORS

The following Chemicals are efficiently and economically handled by DRACCO Pneumatic Conveyors in Water Works and Sewage Treatment Plants located in all parts of the country. Alum, lime, soda ash, bauxite, ferrous sulphate, pebble lime, activated carbon, and ammonium sulphate.

A PARTIAL LIST OF *DRACCO* INSTALLATIONS

FRIDLEY FILTRATION PLANT.....Minneapolis, Minn.
DALECARLIA FILTRATION PLANT.....Washington, D. C.
MUNICIPAL WATER SOFTENING PLANT.....Sandusky, Ohio
MUNICIPAL WATER SOFTENING PLANT.....Ames, Iowa
LEMIEUX ISLAND FILTRATION PLANT.....Ottawa, Ontario
MAHONING VALLEY SANITARY DISTRICT.....Niles, Ohio
MUNICIPAL WATER PLANT.....Columbus, Ohio
MUNICIPAL WATER PLANT.....Peru, Ill.
U. S. GOVERNMENT FILTRATION PLANT..Ft. Leavenworth, Kans.
MUNICIPAL WATER PLANT.....Hamilton, Ohio
BACHMAN WATER PURIFICATION PLANT.....Dallas, Texas
MUNICIPAL WATER PLANT.....Richmond, Va.
MUNICIPAL WATER PLANT.....Springfield, Ill.
MINNEAPOLIS-SAINT PAUL SANITARY DISTRICT..St. Paul, Minn.
(Sewage Disposal Plant, 2 installations)
MUNICIPAL WATER PLANT (2 installations)....Milwaukee, Wis.
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MUNICIPAL SEWAGE DISPOSAL PLANT.....Denver, Colorado
MUNICIPAL WATER PLANT.....Covington, Ky.
MUNICIPAL WATER PLANT.....Ann Arbor, Mich.
CLAYTON SEWAGE DISPOSAL PLANT.....Atlanta, Ga.
SEWAGE DISPOSAL PLANT (2 installations).....Cleveland, Ohio
MUNICIPAL WATER PLANT.....Benton Harbor, Mich.
MUNICIPAL WATER PLANT.....Warren, Ohio
MUNICIPAL SEWAGE DISPOSAL PLANT.....Detroit, Mich.
MUNICIPAL WATER PLANT.....Flint, Mich.
CITY OF TORONTO WATER WORKS PLANT.....Toronto, Ontario
WATER FILTRATION PLANT.....Ottawa, Kansas
EL RENO WATER TREATMENT PLANT.....El Reno, Oklahoma
CITY OF GRAND FORKS.....Grand Forks, North Dakota
MUNICIPAL SEWAGE PLANT.....Pittsburgh, Pennsylvania
WATER FILTRATION PLANT.....Wichita, Kansas
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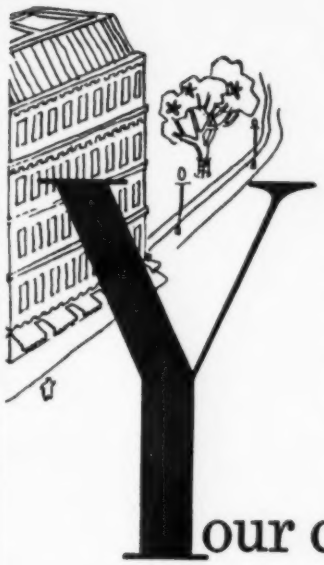
Write for Bulletin No. 526

DRACCO CORPORATION

4079 E. 116th St., Cleveland 5, O. • New York Office, 130 W. 42nd St.

PNEUMATIC CONVEYORS • DUST COLLECTORS



Altitude Control Valves by Golden-Anderson faithfully maintain a 3" to 12" water level variation, and are designed to assure unfailing water control under all climatic conditions. A special needle valve adjustment permits proper valve speed to suit any operating requirements, while inherent water cushioning design minimizes surge and water hammer problems. These valves may be arranged with auxiliary equipment to perform a variety of water control services. Write for complete information today.

Your community...

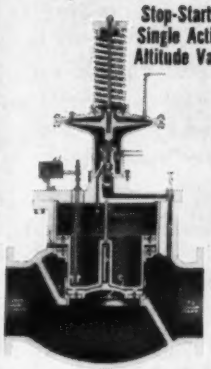
needs the safety and lasting dependability of

GOLDEN-ANDERSON

POSITIVE CONTROL

Valves

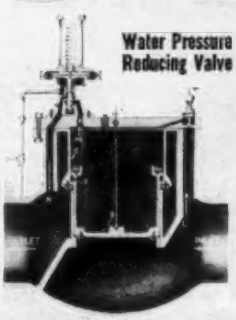
Balanced Controlling Float Valve



Stop-Starter Single Acting Altitude Valve

GOLDEN-ANDERSON VALVE Specialty Co.

Keenan Building • Pittsburgh 22, Pa.



Water Pressure Reducing Valve

Stop Check Single Acting Altitude Valve

Up in North Adams, Mass., a man died in December. I would have said an old man died, but at 93 years, Frank Blake was still active in engineering and surveying. He didn't retire from the City Engineer's staff until he was 85 and at 80 he invented a device for locating lost sewers. He had invented a pipe locater twenty years before that.—A couple of years ago I wrote a piece about him here, and how he was still going strong, and I ended something like this—"There's a mark for you young fellows to shoot at." I still think so.

* * *

When Gordie Weist (Chemist of Albright and Friel, Cons. Engrs. of Philly) was talking at the meeting of the Penna. Sewage and Industrial Waste Assn. in State College, Pa., last August, he started out by saying, "If I can't convince you, at least maybe I'll be able to confuse you." 'Stoo bad more speakers can't be as convincing as Gordie, with as little confusion as he exhibited.

* * *

In an issue of the Indiana Health Dept. News Bulletin, Dr. Thurman Rice called author John Gunther to task for writing somewhat disparaging remarks about Indiana in the book "Inside U.S.A." Dr. Rice was particularly displeased with Gunther's remarks that Indianapolis has the "second ugliest monument in the world" and that he (Gunther) couldn't understand how the "wastes of Indiana could have produced a man like Willkie."

Being the great-great-grandson of the founder of Indianapolis (although I'm a "Sucker" from "Uncle Joe's Town") I couldn't resist sending Rice's editorial to Gunther, who sent a polite note thanking me for my interest. Mr. Gunther's note was addressed to me in care of *Water & Sewage Works*, Indianapolis, Ind.—Someone there must have known that *Water & Sewage Works* is a magazine, even if Mr. Gunther didn't, because the letter was forwarded to me in New York.

* * *

PUZZLE CONTEST No. 2
What Does It Say?

(To whom and by whom was it written—Hint, there are three persons who are not eligible to compete—the fellow who gave this to me and the two persons concerned.)

* * *

Don't let Harry Jordan (Exec.-Secy. A.W.W.A.) ever tell you that he doesn't know anything about water chlorination. Out Indianapolis way they tell the story about Harry getting a good-

QUALITY
Control
OF
BUFFALO METERS
PAYS OFF
FOR YOU!

A portion of factory test department for large meters.

● Quality control of materials and manufacturing assures top performance of Buffalo Meters that will in turn mean long-term uninterrupted service for you!

Quality control culminates in the individual accuracy calibration and hydrostatic pressure test. Accuracy that endures is attained by a balance between precision of measurement and freedom of action.

These fine-quality meters have won a majority of the many accuracy tests in which they have been entered—ample proof of how quality control pays off for you when you equip with *American* or *Niagara*.

Buffalo Meter Company manufactures the *American* Meter (bronze casing) for soft or filtered water, and the *Niagara* Meter (galvanized-iron casing) for most hard water requirements.
Write for descriptive booklet!

AMERICAN BRONZE CASE
NIAGARA GALV. IRON CASE
WATER METERS
BUFFALO METER COMPANY
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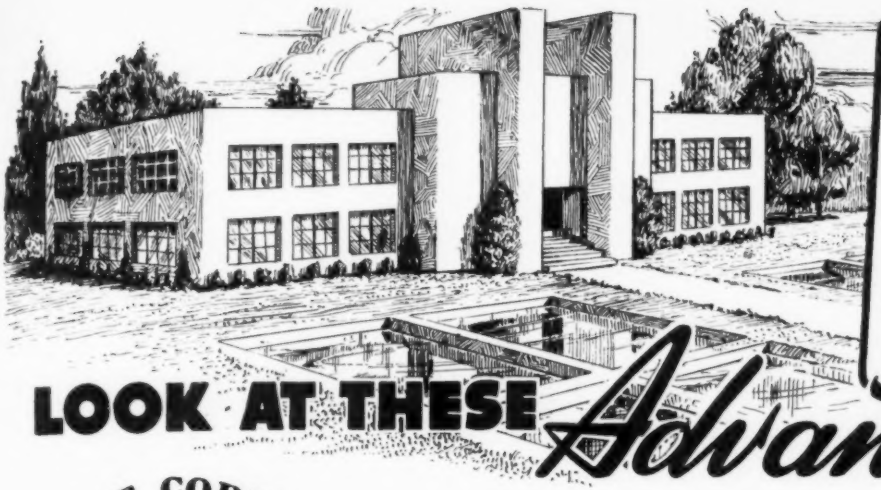


Write for New Booklet

This book contains information, charts, diagrams, etc., as well as general information on the advantages to be obtained with FERRI-FLOC.

TENNESSEE CORPORATION
Grant Building Atlanta, Ga.

LOOK AT THESE *Advantages...*

TENNESSEE CORPORATION
Ferri-Floc
FERRIC SULPHATE

As Summarized By a Prominent Professor of Chemistry and Water Consultant of a Large University

Coagulation is effective over a much wider pH range than with other coagulants. Color flocs may be formed in the very acid range. On the other hand, true hydrated ferric oxide flocs may be formed at pH 9-10, or even higher, for the removal of turbidity and manganese.

The time required for floc formation, conditioning, and settling is in most cases considerably shorter than that required for other coagulants.

Filter runs have been markedly increased in most cases.

Manganese is successfully removed at pH values above 9.

Effluents may be produced which are exceedingly low in both iron and aluminum.

Ferric floc does not seem to stick to sand grains to form mud balls, and is subject to less "breaking through" on the filters.

TENNESSEE CORPORATION
Atlanta, Georgia — TENNESSEE CORPORATION — Lockland, Ohio

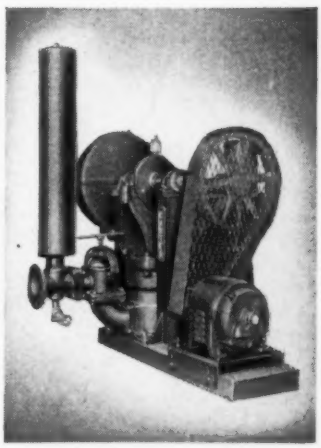
SLUDGE CARTER PUMPS

1848—

The Discovery of Gold in California.

1948—

The Discovery by engineers and contractors that greater savings can be effected by choosing CARTER sludge pumps than those of any other manufacturer. Savings in operation are reflected in savings in dollars when your sludge pump is a rugged, tough, dependable CARTER model. The proof of the product is in the performance — specify CARTER.



RALPH B. CARTER COMPANY
HACKENSACK, N. J.
53 Park Place, New York 7, N. Y.

ly dose of chlorine gas when something went wrong. — (Yes, thirty years ago I can remember that chlorine leaks were not so infrequent as they are today)— Well, on that day, Harry had got a snootful (of chlorine, of course) and was exhibiting all of the symptoms of an overdose of chlorine. Co-workers gathered around to comfort, console, and help. Always the independent, Harry would have none of the proffered assistance and wandered off through the woods to be alone.

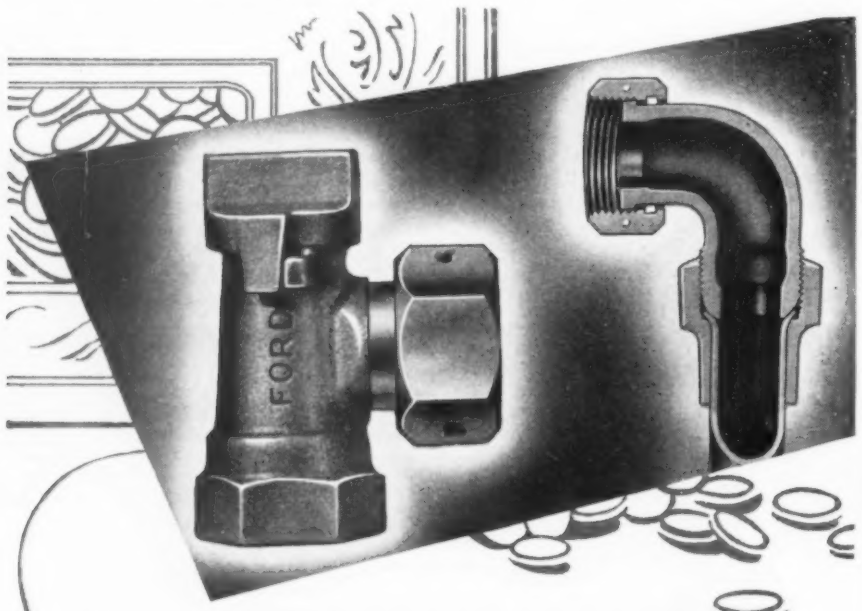
What Harry didn't know was that his worried co-workers followed him at a discreet distance. Wonder if he knows yet?

* * *

Want to make something of it, if you are superstitious? When we moved from Buffalo to New York just five years ago, it was just a few weeks before Easter Sunday. On Easter, we had our first visitors in our new home. First came the TOMBS; shortly, thereafter, the ANGELLS arrived (Jim and Ivy, that is!)

* * *

It doesn't seem like five years since I gave up the test tube for the typewriter, but this is the sixth Reference & Data Issue I've lost sleep over.— Hope you like it! V.T.Y., Doc.



FORD RINGSTYLE VALVES and COUPLINGS HIT THE JACKPOT

Never, until Ford's Ringstyle Valves and Couplings were invented, was it possible to set a meter in such close quarters or with so few fittings.

The secret lies in the Ringstyle construction—a threaded coupling nut is pushed over a machined flange from the face side and then a bronze ring is driven into a groove in the nut and behind the flange, making a strong shoulder. These ingenious fittings really hit the jackpot when it comes to saving time and labor costs.

For complete information, send for Ford Catalog No. 46.

FOR BETTER WATER SERVICES

FORD

THE FORD METER BOX COMPANY, INC.
Wabash, Indiana

[Overflow from the main section]

Wrought Iron Pipe Film

The A. M. Byers Co. has sponsored a film called "Eternally Yours," which covers the history of wrought iron since its early development down to its present-day processes of manufacture, physical properties and usage. This sound film is available for showings in section meetings, or elsewhere.

Requests for showings of this picture may be made either to the sponsor, A. M. Byers Co., of Pittsburgh, Pa., or through the Modern Talking Picture Service, 9 Rockefeller Plaza, New York 20, N. Y.

Winchester, Va., Going Ahead With Sewage Plant

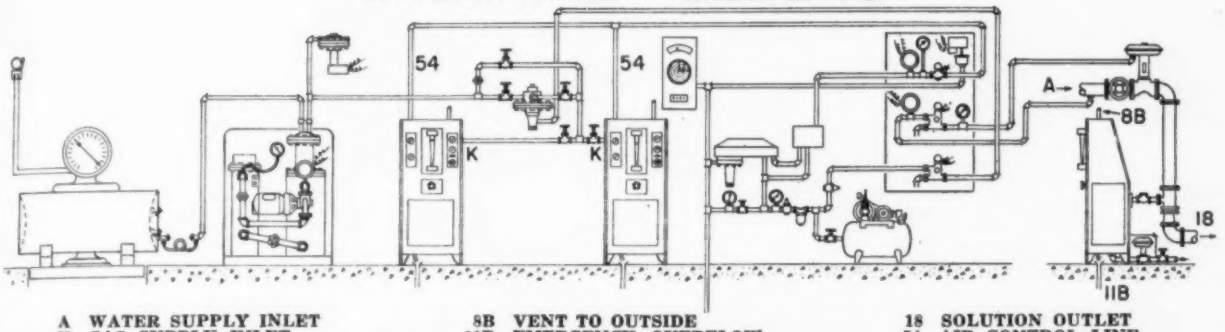
Financed With 1.5% Bonds

In support of Virginia's stream clean-up program Winchester, located on the Shenandoah in the famed Valley of Virginia and home of Senator Harry Flood Bird, is going forward with a sewerage project estimated to cost about $800,000 at present prices. The work will include 17,000 ft. of lateral sewer extensions; 21,000 ft. of water main extension; 10,000 ft. of 24- to 27-inch main outfall sewer; treatment plant providing separate sludge digestion and two-stage high-rate filters.

The treatment plant, designed by Alexander Patten Associates of New York, will have an average design flow of 2.4 million gallons daily, and a design "population equivalent" of 30,000 people. The "equivalent population," about twice actual population, results from the additional organic load produced by apple by-products plants and also by wastes from one of the city's woolen mills.

A permissive referendum to issue bonds for the construction of the work was approved by a seven to one majority. The bonds have been sold at the unusually low rate of 1.5%.

HIGH CAPACITY–AUTOMATIC PROPORTIONING PROGRESSIVE AND RETROGRESSIVE CHLORINE FEEDERS



A WATER SUPPLY INLET
K GAS SUPPLY INLET
8B VENT TO OUTSIDE
11B EMERGENCY OVERFLOW
18 SOLUTION OUTLET
54 AIR CONTROL LINE

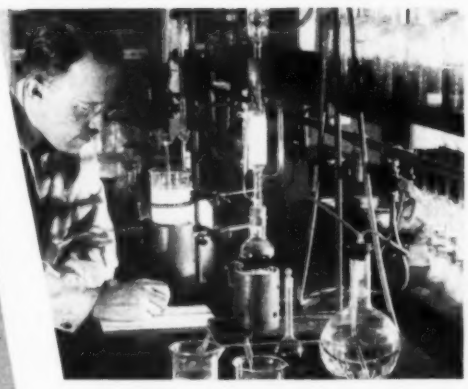
FUNCTIONAL POINTS OUTLINING OPERATION OF TYPICAL INSTALLATION AS PER SKETCH SHOWN ABOVE.

1. Automatic progressive proportioning—ratio 20 to 1.
2. A republic air-operated transmitter is a controlling factor.
3. Accurate automatic proportioning control available with Venturi tube, pitot tube, orifice plate, parshall flume, large conduit and various types of Weirs.
4. Modulator is required where open irregular structures are used.
6. Operation either manual or automatic proportioning obtained by a one-half turn of the selector valve.
8. Chlorine gas and liquid are mixed in a **high** vacuum equal to an 8" mercury vacuum.
9. The injector is the power plant.
12. Description of complete operating cycle—600lb to 12,000lb of chlorine gas applied.
13. How we obtain an operating ratio of 20 to 1 and why.
14. Chlorine gas is automatically proportioned as the liquid flow increases or decreases.
15. Automatically operated safety features.
16. Alarm at main office by using different colored lights.
17. Necessity for continuous supervision by operator or attendant is eliminated.

AN UNUSUAL ACCOMPLISHMENT YOU SAY—SOME SAY UNBELIEVABLE. IF YOU WILL WRITE US AT ADDRESS GIVEN BELOW, WE WILL BE PLEASED TO SEND YOU A FULL DESCRIPTION OF THE OPERATION OF THIS UNIQUE CHLORINE CONTROL EQUIPMENT INSTALLATION.

EVERSON MFG. CORP.
233 W. HURON STREET
CHICAGO 10, ILLINOIS

LABORATORY-TESTED *HOURLY!*



Gold Bond

HYDRATED AND QUICK **LIME**

Simplify your water purification problems with one of these TESTED lime products. Guaranteed uniform high calcium oxide content. Write for information on your requirements.

NATIONAL GYPSUM COMPANY, BUFFALO 2, N. Y.

High Calcium Lime Plants at Kimballton, Va., Bellefonte, Pa., and York, Pa.

General Motors Has Industrial Waste Control Committee

The General Motors Corp. inaugurated an Industrial Waste Control Committee in 1944 and since that time has developed considerable information toward relieving waste pollution from General Motors industrial plants. In the intervening years many meetings have been held in various plants at which outstanding personalities in industrial waste treatment have spoken.

The corporation has also retained the services as consultants of George E. Barnes, Professor at Case School of Applied Science, Cleveland; Don E. Bloodgood, Professor at Purdue University; and Edward F. Eldridge, Professor at Michigan State College, Lansing.

A.P.H.A. Water Supply Committee Report

The Committee on Water Supply of the American Public Health Association has issued its annual report on Water Supply in the Feb. 1948 issue of the *Journal of the American Public Health Association.*

While the Committee did not give detailed discussions of the new developments in water supply, it did draw attention to current trends having bearing on the potability of water supplies. Among the trends are better control of chlorination, and the distinguishing between free residual chlorination and combined residual chlorination, with the various tests for that purpose. The effect of pH on the chlorine residual is also discussed. Among the other items covered are the application of chlorine dioxide, the laboratory technic for its determination; research on the application of fluorine to water supply for the prevention of dental caries; and high rate filtration.

Another subject given consideration was the sodium content of potable water and its effect on disease; also the effect of nitrate content of water and cyanosis.

The committee calls attention to the fact that definite data are not yet available on the possible toxic effect of ingredients of potable waters incidental to algae growths. It likewise draws attention to the need for a more precise and definite indicator of bacterial pollution than the heterogeneous coliform group.

The committee also points out the need for further research, particularly in the field of laboratory technic and says that insofar as bacterial warfare is concerned, it has been indicated that pollution of public water supply by intestinal pathogens is not satisfactory for this purpose.

New Method for Disinfecting Water

In two recent patents, granted by the U. S. Govt., G. P. Ham and Dr. R. B. Barnes, research chemists of the American Cyanamid Co., set forth claims for a new system of treating water to remove bacteria. No longer will there be any chlorinous or chlor-phenol tastes and odors if this proposed ideal is adopted on a wide scale.

The method consists of removing bacteria by electrically precipitating them on sand or other di-electric material filter. The filter consists of a cylindrical rubber lined tank containing sand separated into layers by thin mats of glass wool. Platinum electrodes, properly spaced, pass an electric current through the sand as the water filters through. The bacteria are precipitated out on the surface of the sand grains and are not killed. Most of them are in the upper layer and can be killed by means desired. (Any comments? Ed.)

A Place for Everything—Everything in It's Place!

IT'S EASY TO WORK WITH THE "FLEXIBLE" "CARRY-ALL" TRAILER

Carries Complete Equipment for Rodding Sewers, Cleaning Lines.

Save time—save labor—save stand-by truck with the new "Flexible" "Carry-All" Trailer. A complete sewer cleaning unit on wheels, always ready for instant service. With a place for everything, the "Carry-All" is easy to keep shipshape at all times. Ezy Rod and Reel stand permanently mounted—rods easily reeled directly from trailer. Why not end that bother of loading a truck, when you've got everything you need and always handy on a "Carry-All"?

"Flexible" equipment is sold exclusively through Jobbers. Buy exactly what you need from them. Illustrated Catalog available. Write for your copy.

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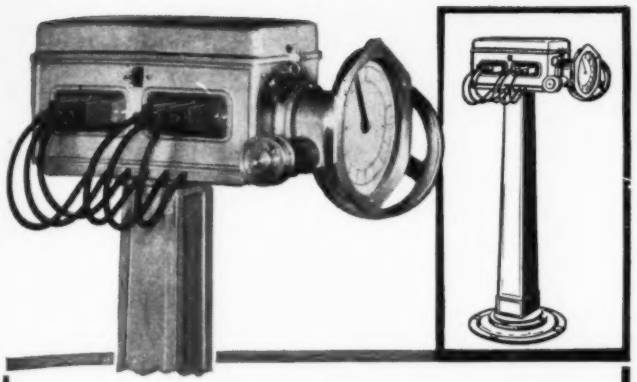

For BALANCED PUMP OPERATION

Specify SELECTROL

SELECTROL closely balances total pump capacity with rate of inflow at each operating level, for either rising or falling level cycle. It handles up to 18 circuits singly or in combination for any float travel up to 40 feet. A large number of circuits can be handled on limited float travel. A feature of SELECTROL is the "Lost Motion Clutch" which balances pumps of unequal capacity without "hunting". AUTOMATIC can help you with your multiple circuit problems.

Write for Bulletin S-7

AUTOMATIC CONTROL Company
1013 University Ave., St. Paul 4, Minn.

5 GOOD REASONS
for buying Blaw-Knox Grating
. . . COUNT 'EM!

1. STRONG electroforged construction for easy erection.
2. MAXIMUM OPEN AREA for light and air.
3. EASY TO MAINTAIN . . . paint reaches entire surface.
4. SELF-CLEANING, no sharp corners to clog.
5. SAFE footing at all times with twisted cross bar.

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BLAW-KNOX ELECTROFORGED STEEL GRATING

SEWER MAINTENANCE ENGINEERS AND PLANT OPERATORS

Stop Odor Complaints
Prevent H.S Corrosion
Cut Maintenance Costs
Control Sewer Insects
Reduce E. Coli Counts

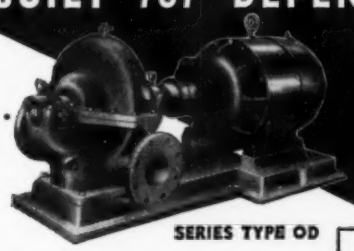
and receive a Non-Septic Sewage, easy to handle, at their plants

all with
UP-SEWER *Cloroben* **TREATMENT**

THE CLOROBEN CORPORATION
15 EXCHANGE PLACE JERSEY CITY 2, N. J.
The Pioneers of Up-Sewer Treatment

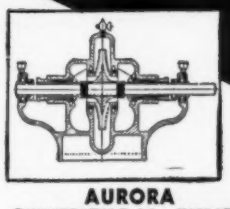
BUILT for DEPENDABILITY

AURORA HORIZONTALLY SPLIT CASE DOUBLE SUCTION SINGLE STAGE CENTRIFUGAL PUMPS *"by Aurora"*



SERIES TYPE OD

The ultimate in design and perfection — type OD pumps — in capacities to 4,000 G.P.M. and heads to 300 ft. — are accessible, compact and lasting. They are widely used for general water supply, for municipalities, industrial buildings and institutions. Also for handling condenser circulating water, liquids in paper mills and distilleries, chemical solutions, oil in oil fields and refineries, irrigation, etc.



AURORA CENTRIFUGAL PUMPS — are also available in HORIZONTALLY SPLIT CASE TWO STAGE DOUBLE SUCTION • SIDE SUCTION • VERTICAL • NON CLOG • SUMP • MIXED FLOW • DEEP WELL TURBINES • and SPECIAL DESIGN.

Write for **CONDENSED CATALOG "M"** *or* **CONSULT SWEETS**

APCO TURBINE-TYPE PUMPS
Here's the Pump for "1001" duties. SIMPLE — only one moving part, the impeller. Capacities to 150 G.P.M., Heads to 600 Ft. Slight change in capacity against drastic head variations.

DISTRIBUTORS IN PRINCIPAL CITIES

AURORA Always PUMPS — **AURORA PUMP COMPANY**
68 Loucks Street, AURORA, ILLINOIS

King's Hydraulics Revised

For years, King's handbook on Hydraulics has been an essential tool to sanitary engineers. The Fifth Edition of this book is now available, completely revised, rearranged and re-edited in accordance with the latest research and practice. The book carries the title "HYDRAULICS," and is authored by Horace W. King, Professor Emeritus of Hydraulic Engineering, University of Michigan, Chester O. Wisler, Professor of Hydraulic Engineering, University of Michigan and James G. Woodburn, Professor of Hydraulic Engineering, University of Wisconsin.

A Fifth Edition of this book on Hydraulics contains the following chapters: Fundamental Properties of Fluids, Principles of Hydrostatic Pressure, Hydrostatic Pressure on Surfaces, Relative Equilibrium of Liquids, Fundamentals of Fluid Flow, Orifices, Tubes, and Weirs, Pipes, Open Channels, Hydrodynamics, Hydraulic Similitude and Dimensional Analysis.

Published by John Wiley & Sons, Inc., 440 Fourth Ave., New York 16, N.Y., the Fifth Edition of Hydraulics by King, Wisler and Woodburn contains 351 pages (5⅝ x 8⅝ in.) and may be obtained for $4.00 a copy.

Hazleton Type "M"

CHECK VALVE

with Built-in Mercoid Motor Shut-off Switch for Automatic Pump Protection

PROTECTS PUMP FROM DAMAGE DUE TO LOSS OF WATER

PREVENTS OPERATION OF PUMP UNTIL PROPERLY PRIMED

Flow switch insures pump against damage in event pump loses water during operation. Mercoid switch is actuated by valve-flap. Switch is connected into no-voltage coil of motor starter.

Write for Further Information

BARRETT, HAENTJENS & CO.

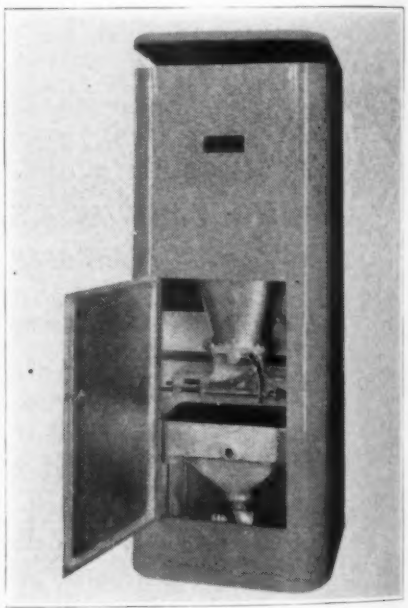
HAZLETON PENNSYLVANIA

MANUFACTURERS' and EQUIPMENT NEWS

400

Dry Chemical Feeders

F. B. Leopold Co., Inc., 2413 W. Carson St., Pittsburgh 4, Pa., is offering a new dry chemical feed machine. The new L P Dry Chemical Feed Machine is contained in a cabinet, within which is a hopper, feed mechanism, drive mechanism, control adjustments, and agitators. Below is a dissolving chamber and the drive mechanism is operated by an electric motor.

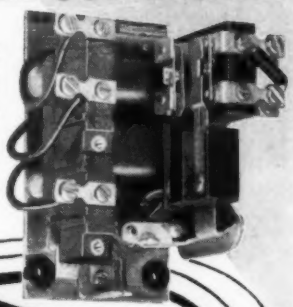


The type L P Dry Chemical Feed Machine comes in three sizes, No. 0, No. 1, and No. 2. The hopper capacity in each case is 5 cu. ft., but the feeding capacity for such chemicals as Hydrated Lime, Soda Ash, Alum, Nuchar, etc., varies and the minimum and the maximum range of the machine varies.

If you desire further information, mail a Reader Service Card containing the above key number.

All Electric-Floatless LIQUID LEVEL CONTROLS

Positive! Dependable! Economical! No floats! No vacuum tubes! No moving parts in liquid. Unaffected by acids, caustics, pressures or temperatures.



B/W

WRITE FOR COMPLETE CATALOG

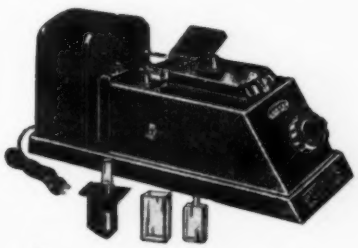
B/W CONTROLLER CORPORATION

2206 E. MAPLE ROAD BIRMINGHAM, MICH

FIRST IN THE FLOATLESS CONTROL FIELD

KLETT SUMMERSON ELECTRIC PHOTOMETER

Adaptable for Use in Water Analysis



Can be used for any determination in which color turbidity can be developed in proportion to substance to be determined.

KLETT MANUFACTURING CO.

179 EAST 87th STREET • NEW YORK 28, N.Y.

3 *Money, Time and Labor Saving Features of*

UNIVERSAL* CAST IRON PIPE

Laid with Only Wrenches →

No Caulking Materials →

No Gaskets. No Bell Holes to Dig. →

*Reg. U.S. Pat. Office

For water supply, fire protection systems, sewage disposal systems, industrial, and irrigation. Flexible.

Dept. C,
THE CENTRAL FOUNDRY COMPANY
386 FOURTH AVENUE, NEW YORK 16, N. Y.

Gentlemen: Send us information and catalog on UNIVERSAL CAST IRON PIPE.

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think of it!

YOU AVOID 80%-90% ON EXCAVATING COSTS BY USING HYDRAUGERS

From an operating trench approximately 3½ ft. by 8 ft., the HYDRAUGER bores clean open holes as far as 225 ft. This modern labor saving machine eliminates costly trenching, pavement damage, backfilling, interference with traffic, etc. Frequently saves 50%-75% on man-hours. Pays for itself within a year and often in a few weeks.

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HYDRAUGER machines are available in 3 different types (3.2 and 5.2 horsepower). Bores 2 in. and 2½ in. hole. Bore-reams 3½ in., 4½ in., 6½ in., 8½ 10½ in., 12½ in., 14½ in. and under certain conditions to 24 in. dia. hole.

Write for Catalog No. 8 TODAY!

HYDRAUGER CORP., Ltd.
681 Market Street
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OPEN HOLE

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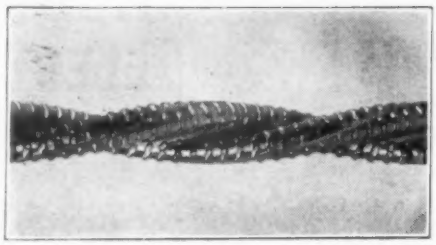
HY-13b

EARTH BORING TOOL

401

New Form Reinforcing Bar

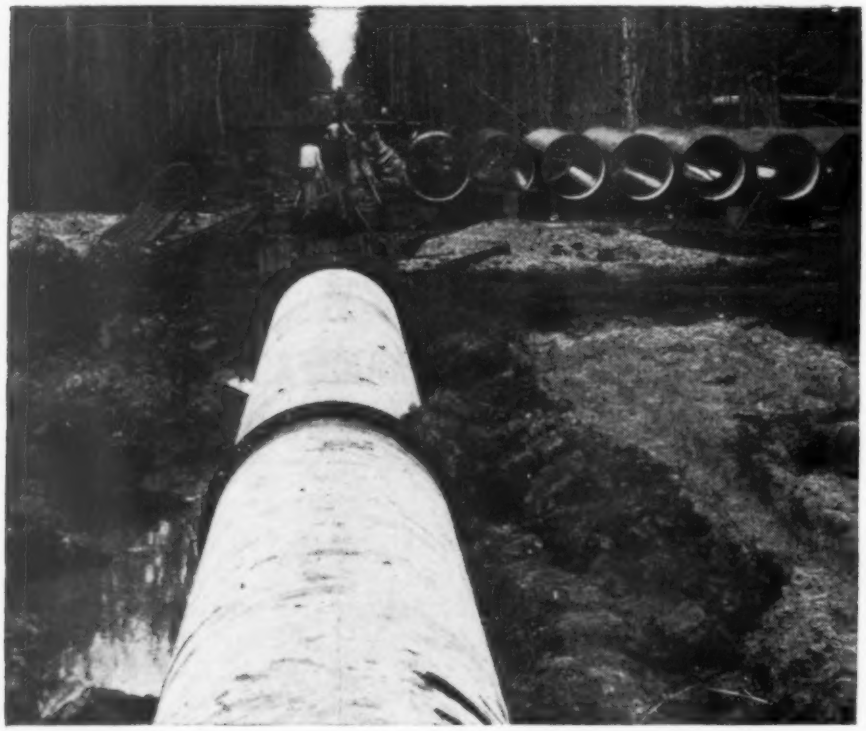


The Webrib Steel Corp. of Danville, Penna., has developed a new form of reinforcing bar, known as the Webrib. This bar, which is a deformed dumbbell shape, has a yield point of 50% higher than normal steel bars. The use of these bars saves one-third the weight of steel in construction and, in addition, a considerable economy in cost is effected by saving of material, handling, transportation, and so forth.

The Webrib engineering data booklet is available for further information from the Webrib Steel Corporation, 120 Broadway, New York 5, N. Y., or may be obtained by using a Reader Service Card.

Alhyrdo, Inc.

A new company name in the field of flocculation has been introduced. Known as Alhyrdo, Inc., and located at 516 N. Charles St., Baltimore 1, Md., the company manufactures a new type of aluminum hydroxide floc producer utilizing the principle of electrolytic dissolution of aluminum. Equipment is of new design and is offered for use in flocculating water for both industrial and domestic use. General Manager of the new company is Al Brumley, formerly of Stuart-Brumley Corp.



Consulting Engineers, J. E. Sirrine Company; General Contractors, Diamond Construction Company and Shepherd Construction Company; pipe supplied by Bethlehem Steel Company.

KOPPERS

BITUMASTIC*

PROTECTS THIS 14-MILE

PIPELINE INTO SAVANNAH

THIS 14-MILE PIPELINE will carry 35,000,000 gallons of water daily to the City of Savannah, and to a large paper mill in the area. It passes through swampland where corrosion is the big problem.

To prevent corrosion from these severe conditions, the exterior of the pipe is protected with Koppers Bitumastic 70-B Enamel. For the pipe interior, freedom from corrosion, rust and tuberculation, and continued high flow coefficients are assured by a glass-smooth spun lining of the same impervious coating.

Specify Bitumastic coatings—proven leaders for over 90 years in protecting metal, concrete and masonry. Write for data on your particular problem.

*Reg. Trade Mark, U. S. Pat. Off.

KOPPERS COMPANY, INC. PITTSBURGH 19, PENNSYLVANIA

Howard A. Marple Named Head of Monsanto's New Central Advertising Dept.

Howard A. Marple, for a number of years editor of the outstanding award winning *Monsanto Magazine* published monthly by Monsanto Chemical Co., has been chosen to direct the new Central Advertising Department recently organized by MONSANTO to coordinate all phases of its advertising and publicity activities.

Within the new Central Advertising Dept. will be an advertising manager for each of the six major divisions of MONSANTO. The managers so far named are R. Allan Gardner (Organic and Phosphate Divs.); Edmund Green (Merrimac Div.); Edmund Kennedy (Plastics Div.); Braxton Pollard (Foreign Dept.).

Howard Marple has been connected with MONSANTO since 1937 as editor of *Monsanto Magazine* and more recently also as director of trade advertising. He is a native of Boston and a graduate of Tufts College (B.S. in chemistry). In World War I he served in the Chemical Warfare Service. He was one of the founders and is a past president of the International Council of Industrial Editors. He is a member of the Industrial Marketing Council of St. Louis; the Advertising Club of St. Louis and the Am. Chemical Society. He has been widely recognized for his work as editor of the highly regarded *Monsanto Magazine*.

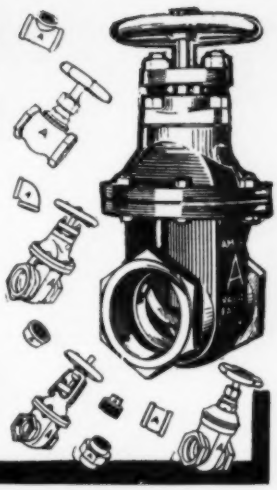
VALVES AND FITTINGS

DEPENDABLE PERFORMANCE SINCE 1901

Iron Body Gate Valves
Iron Body Check Valves
Iron Body O.S.Y. Valves
Iron Comp. Flanges
Iron Flange Unions

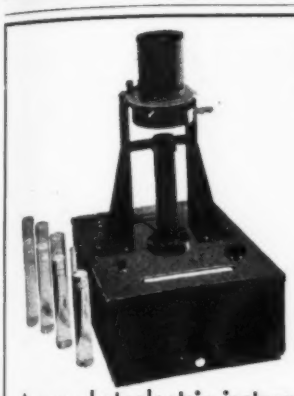
"A" TRADE MARK

Brass Pipe Fittings
Brass Gate Valves
Brass Globe Valves
Brass Check Valves
Brass Radiator Valves
Brass Gas Cocks



AMERICAN VALVE MFG. CO. Coxsackie, N. Y.

N. Y. OFFICE & WAREHOUSE: 37-25 32ND ST., LONG ISLAND CITY 1, N. Y.



LUMETRON

Photoelectric Colorimeter Model 450 for Nessler Tubes

A new photoelectric instrument of high accuracy for the measurement of color and turbidity, as well as for all analytical colorimetric tests in the examination of water and sewage.

- Turbidity tests in terms of APHA (ppm) scale.
- Color tests in terms of APHA (mg Pt) scale.
- Suited for determination of ammonia nitrogen, nitrate, nitrite, and for all other colorimetric tests according to APHA "Standard Methods."
- Operates with all "low form" Nessler tubes 32 mm O.D., 200 mm high. Requires no matched sets of tubes.

LUMETRON Mod. 450 eliminates the uncertainties of visual matching methods. It requires no permanent color standards and furnishes reproducible results independent of individual judgment and of light conditions.

Write for literature to

PHOTOVOLT CORP.
95 Madison Ave. New York 16, N. Y.

Write also for literature on

PHOTOVOLT Line-Operated pH METER

GEOPHONE locates leaks without digging

"Registered in U. S. Patent Office"

Complete Geophone outfit consists of connecting tubes, two Geophone discs, headpiece, and heavy leather-covered carrying case with shoulder strap. **$85**

When the discs are placed on the ground (as pictured) the Globe Geophone detects by sound the exact location of leaks in water or steam pipes. Acclaimed for its accuracy, the Globe Geophone has become indispensable in Public Works Departments and Industrial Plants throughout the United States.

● MONEY-SAVING

You save the costly expense of extensive surveys, excavating and replacing. The inexpensive Globe Geophone pays for itself many times over in money saved.

● ACCURATE

Operating on the principle of the seismograph, the Globe Geophone is without question the most sensitive and reliable instrument for detecting underground leaks.

Pipe Finders . . . $110
Dipping Needles . . . . . $ 20
Pipe Phones . . . $3.70

Globe Phone Mfg. Corp. Reading, Mass., U. S. A. Dept. F

Manufacturers of Geophones Since 1918

The SEWER SCOOTER

(U. S. Patent 2,198,823)

THE ONLY 100% SELF-PROPELLING SEWER CLEANER

THE FLUSHING ACTION IS APPLIED BY COLLAPSING THE SHIELD.

A patented feature

No Preliminary Work— Gets Results in a Hurry

SEWER SCOOTER MODEL A-3, BRONZE

Standard machines stocked 8 inches to 96 inches—For sale, rent or contract.

J. C. FITZGERALD

Box 289-A
Coral Gables 34, Fla.

Portland Cement Association News

Announcement has been made by Frank E. Sheets, President of the Portland Cement Assn., of the election of Carl D. Franks as Vice President for Promotion. Mr. Franks has been a member of the association's staff since 1916 when he became District Engineer in charge of the Indianapolis office.

Subsequent to the election Mr. Franks as Vice-President announced that A. M. Davis of Lansing, Mich., was appointed Manager of Midwestern Offices of the Association. Mr. Davis succeeds to the position made vacant by Mr. Franks' election.

To succeed Mr. Davis as District Engineer of the Association's office in Lansing, appointment was made of J. Gardner Martin of Detroit.

Link-Belt Appointments

The Link-Belt Company has expanded its service to its customers in three new areas by appointing Sales Representatives to head up the respective sales offices. In the new Link-Belt sales office in Louisville, Ky., Mr. Emmart LaCrosse, Jr., formerly district sales engineer at Indianapolis, will become the head. In Wilmington, Del., William H. Kinkead will be in charge of the new office, and in Charlotte, N. C., Thomas H. Appleton will be District Manager.

R. V. Baker Named Vice-Pres. Omega Machine Co.

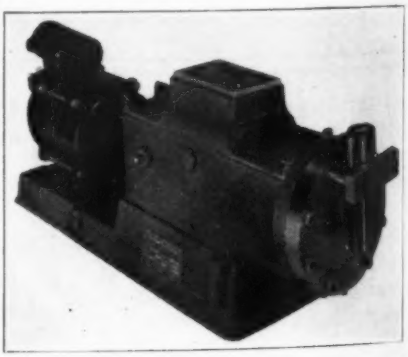
Ralph V. Baker, Assistant Secretary of the Omega Machine Co., has been named Vice-President, according to an announcement by President L. E. Harper. Mr. Baker joined Omega in 1935, before the company

became an affiliate of Builders Iron Foundry. His first service was as Development and Experimental Assistant to the late E. E. Harper, founder of the company. He was subsequently named Chief Draftsman and in 1942 became General Manager. When the firm was acquired by Builders Iron Foundry and the factory moved to Providence, he was made Assistant Secretary.

402

Pulsafeeder Proportioning Pumps

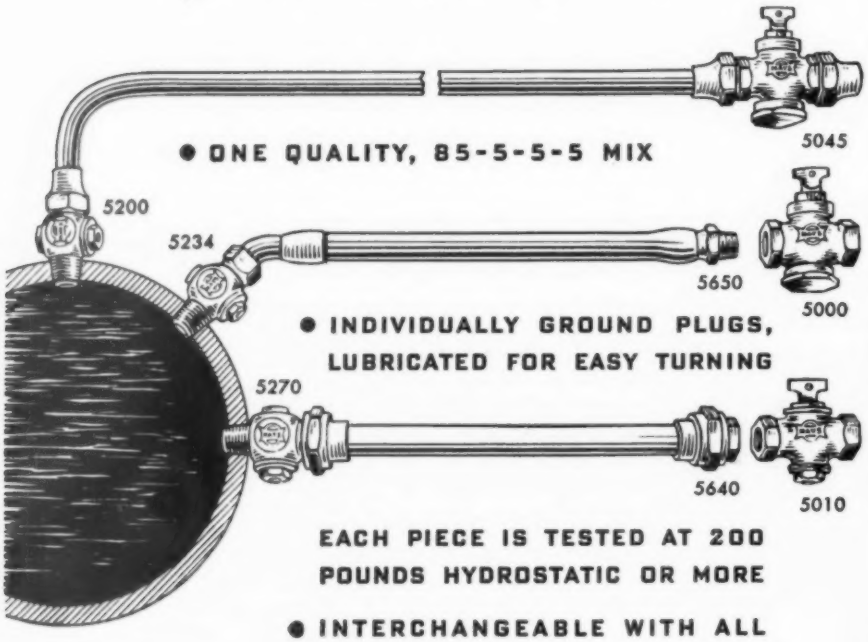
The Lapp Insulator Co., Inc., 77 Maple St., LeRoy, N. Y., has announced a new line of Pulsafeeder



Proportioning Pumps, designated as CP models. The new series represents a completely new design, which provides greater capacities, high pressure operation and rugged assembly

HAYS

CORPORATION and CURB STOPS
for Every System



HAYS
COPPER · BRASS · LEAD · IRON
WATER WORKS PRODUCTS
HAYS MANUFACTURING CO., ERIE, PA.

for accurate feeding. The pump is suited to a wide range of applications and may be obtained in four sizes with the smallest range from 0 to 11 gallons per hour and the largest range up to 330 gallons per hour. The Pulsafeeder Pump is a combination piston diaphragm unit, the piston reciprocating over a measured stroke thereby displacing oil, which in turn pulsates a diaphragm.

If you desire further information, contained in Bulletin No. 262, just enter the above key number on a Reader Service Card and mail.

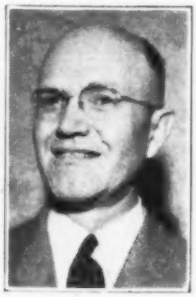
403

Electronic Interval Timer

Photoswitch, Inc., 77 Broadway, Cambridge 42, Mass., has announced the Photoswitch Electronic Interval Timer, Series 30, designed for machinery and process control. It has a range of 0.03 seconds to four minutes and is recommended for the control of chemical feeders, and conveyors, among other equipment.

If you desire Bulletin PC478 which describes this equipment, enter the above key number on a Reader Service Card and mail.

J. M. Palmer, President, NCPMI

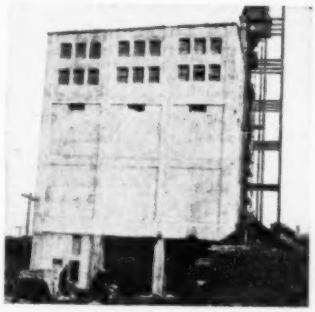


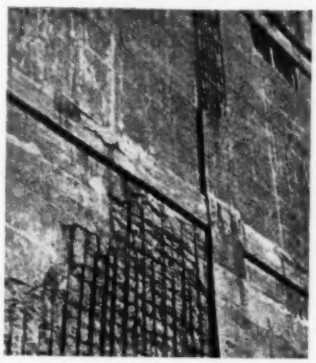
At the annual meeting of the National Clay Pipe Manufacturers, Inc., held in Sioux City, Iowa, early in March, John M. Palmer, sales manager of the Lee Clay Products Co., of Clearfield, Ky., was elected to the presidency of the organization.

CEMENT GUN COMPANY

"GUNITE" CONTRACTORS

GENERAL OFFICES—ALLENTOWN, PENNA., U.S.A.





CONCRETE BUILDING REPAIRED WITH "GUNITE"

This concrete structure is the breaker building at Cayuga Rock Salt Company, Myers, N.Y. The concrete was badly disintegrated due to infiltration of salt and resultant action on the reinforcing steel, as evidenced by the lower close-up photo.

The disintegrated concrete was chipped away and the entire area washed and sandblasted. Mesh reinforcement was placed and anchored into place with expansion bolts. Deeply chipped areas were first filled with "GUNITE" and then a 1½" minimum layer of "GUNITE" applied over the entire area. The finished job is shown in the upper photo.

The concrete slab roof of the building was similarly repaired with "GUNITE" on both sides of the slab. Sufficient concrete was first chipped away so that the ultimate reintegrated slab would be the original thickness, thus preventing an increase in the dead load.

"GUNITE" is impervious and will not permit infiltration of salt brine to attack the steel. Its use as armor against sea water and conditions above described is widespread, and its success remarkable.

Write for our free 72-page descriptive bulletin C 2300

MANUFACTURERS OF THE "CEMENT GUN"

PORTABLE GATE OPERATOR

1½ HP. gasoline engine and flexible shaft drive. Variable speed 20 to 80 RPM. Full reverse by lever control. Automatic centrifugal clutch picks up load after engine is started. Revolution counter. Weight of head only 30 lbs. Only one gate key for all gates. A complete power wrench for any power use.

PAYNE DEAN & CO., MADISON, CONN.

Bailey Meter News

The Bailey Meter Co. of Cleveland, Ohio, has announced an expanding of its staff of Field Engineers. The following assignments have been made: H. E. Bitler to the Buffalo Office, M. J. Bonnette and R. W. Hooper to the Houston area, J. W. Cunningham to the Atlanta Office, G. H. Hilston to the Cleveland District, J. O. Meals to St. Louis, and L. W. Meier to the Los Angeles office.

The company also transferred R. B. Ritschard from the Cincinnati office to the Cleveland District office. J. R. Brennan, J. A. Hays, and R. P. Marche will remain at the Cleveland Main office, attached to the Engineering, Research and Sales Departments respectively.

404

Chemical Cleaner and Deodorizer

Oakite Products, Inc., 173 Thames St., New York 6, N. Y., have announced a new chemical material, possessing a dual ability to clean and deodorize in one single operation.

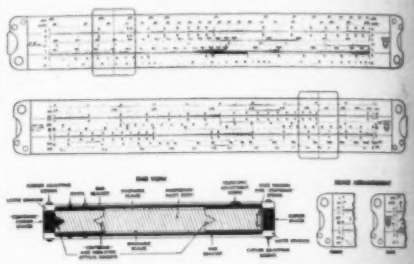
The new material, known as Oakite Di-Sanite, is a finely divided, free-flowing, mildly alkaline powder, readily soluble in hot or cold water, with pronounced wetting, penetrating and detergent characteristics, as well as deodorizing.

If you desire further information, enter the above key number on a Reader Service Card and mail.

405

All-Metal Slide Rule

Pickett & Eckel, Inc., 1111 South Freemont Ave., Alhambra, Calif., have introduced a new All-Metal Ortho-Phase Log Slide Rule. This new rule is non-warping, because it is of all metal construction, assuring

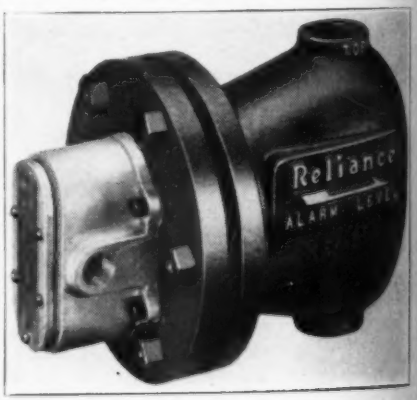


enduring accuracy. It is manufactured to a precision tolerance of 0.001 inch and the slider does not stick, bind or wobble. The surface of the entire rule is unaffected by constant cleaning.

Should you desire further information, just mail a Reader Service Card on which is entered the above key number.

406

Liquid Level Alarm

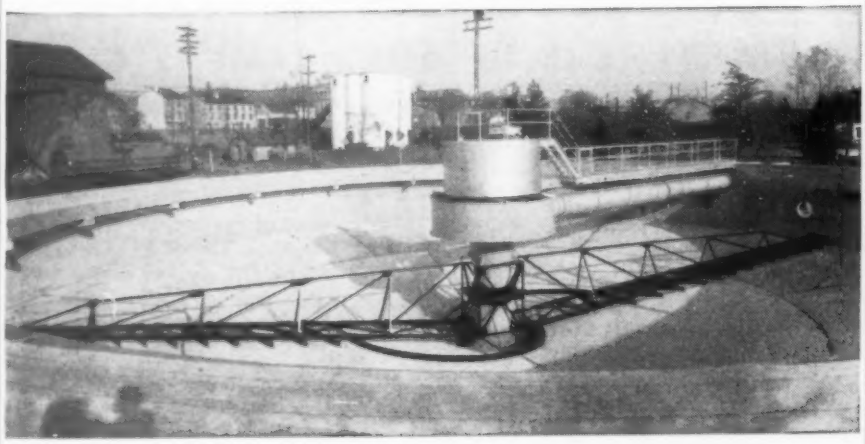


The Reliance Gauge Column Co., 5902 Carnegie Ave., Cleveland, Ohio, has developed and is manufacturing a new device to set off warning signals, known as the "Levalarm." The device can be used on pressures up to 250 pounds. This alarm is sensitive to the slightest water level variations.

HARDINGE

Heavy-Duty

Clarifiers



Above is shown a 110-foot diameter Hardinge Heavy-Duty Center-Pier Clarifier in a municipal installation. This clarifier is capable of removing silt from 8000 gallons per minute of chemically treated river water. Hardinge Clarifiers are simple in construction and built for long life. Notice the combination type scrapers—part cut-flight scrapers and part double-spiral scrapers (near the center discharge) which remove settled solids rapidly from the tank bottom.

Another unusual feature is the Hardinge "Auto-Raise" scraper mechanism, which permits operation during heavy overloads without danger of scraper breakage.

HARDINGE

COMPANY INCORPORATED

YORK, PENNSYLVANIA — 240 Arch St. • Main Office and Works

NEW YORK 17—122 E. 42nd St. • 205 W. Wacker Drive—CHICAGO 6

SAN FRANCISCO 11—24 California St. • 200 Bay St.—TORONTO 1

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Please fill out a separate coupon for each item on which you desire information. This will greatly facilitate handling.

Key number is at head of each new product, item of equipment, catalog or bulletin, described in this issue.

Enter key number in box.

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NOTE—

These cards are to be used only for items described in THIS ISSUE

WATER & SEWAGE WORKS

Will see that you receive information desired.

AN ADDITIVE AND STRENGTHENER OF ASPHALTIC COMPOUNDS

NERVA-SEAL

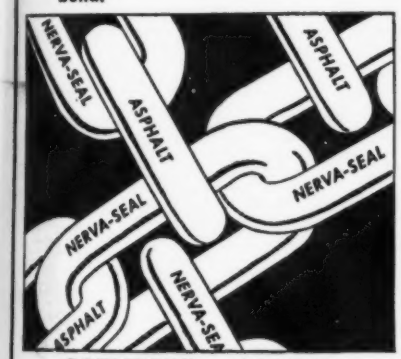
TO PREVENT GROUND WATER INFILTRATION AND PIPE LEAKAGE DUE TO FAILURE OF BITUMINOUS JOINT BOND

NERVA-SEAL liquefies and blends easily with most asphalts and tars, extending their resistance to hot or cold weather. Where asphalt is weakest, Nerva-Seal gives it unprecedented new strength and durability by imparting to the compound its peculiar retractability, better cohesion and resilience. Indispensable for stability in fill construction.

ASPHALT + NERVA-SEAL

RESULTS IN:

- Unsurpassed high impermeability to water, vapor pressure, condensation.
- Non-cracking at sub-zero temperatures, non-flowing at highest summer temperatures.
- Continued better tack and adhesion.
- Permanent maintenance of joint bond.



NERVA-SEAL

THE NEW THERMOPLASTIC CONDITIONER FOR ASPHALT AND TAR

BLEND Nerva-Seal with Asphalt—Achieve durable sealing and leakproofing, high impermeability, imperviousness to impact, weather, corrosions.

•HIGHLY EFFICIENT •BUDGET-WISE •PERMANENT

Write today for NERVA-SEAL fact sheets and PROVEN TEST RESULTS, find out how to plan more economically the NERVA-SEAL way.

RUBBER & PLASTICS COMPOUND CO., INC.
30 ROCKEFELLER PLAZA, NEW YORK 20, N.Y.
Distributors of NERVASTRAL J. F. Joint Filler, and SEAL-PRUF, the Impermeable Sheeting

If you are interested in liquid level alarms, enter the above key number on a Reader Service Card and mail, for further information.

407

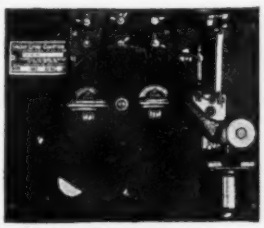
Waterproofing Admix for Concrete

The American Bitumuls Co., 200 Bush St., San Francisco 4, California, is now marketing an integral waterproofing admix for concrete, known as Hydropel.

Hydropel is a brown, slightly viscous liquid which consists of an aqueous suspension of colloidal asphalt, developed for use in cement, concretes and mortars. When added at the rate of one and one-half gallons per sack of cement, Hydropel is effective as an integral admix, used to reduce water absorption and a subsequent reduction in expansion and contraction of the concrete. It improves the dispersion of cement, and also increases the ability of concrete to absorb shock without breaking.

For further information on this product, just enter the above key number on a Reader Service Card and mail.

ROTO-TROL



"A"

TYPE

Type "A" Roto-Trol operates from the pressure in the pump discharge line, at the pump house. The electric time delay prevents starting or stopping due to surges. It also protects deep well turbines from damage due to back-spin . . . *Type "A" Multi-Pump Roto-Trol* is designed for two or more pumps discharging into the same discharge pipe. Adjusted time delays compensate for additional pumping friction and surges when second pump starts or stops.

WRITE FOR PERFORMANCE AND ENGINEERING DATA

Water Levels Control Division

HEALY-RUFF COMPANY

783 HAMPDEN AVE., ST. PAUL 4, MINN.

WEEDS are OUT at water works installations

Dolge SS Weed Killer can be used to advantage in all areas containing water works installations, where no vegetation whatever is desired. Sprinkled on, in economical dilution, this formula works down to the roots, so that it destroys not only surface growth but the entire vegetable structure . . . particularly important where stand pipe locations must be considered. Keeps the growth down around high reservoirs; with proper application there is no danger that the solution will work into storage facilities.

E. W. T. Selective Weed-Killer

(2-4-D) maintains your landscaping but eliminates noxious weeds. Non-injurious to most grasses, E.W.T. aids in keeping a "well-groomed" appearance to the grounds around your pumping plant, purification plant, storage facilities and administration building.

Both DOLGE WEED KILLER and E.W.T. SELECTIVE WEED KILLER have their place in your ground maintenance program. They forestall hours of expensive labor devoted to weeding by "going to the root" of the weed problem.

Complete literature will be sent on request. If you have an unusual weeding problem, we shall be glad to suggest specific methods for dealing with it.

The C. B. DOLGE CO.
WESTPORT, CONNECTICUT

Southern Alkali Completes $10,000,000 Chlorine Plant

Completion of a $10,000,000 program to convert a war surplus magnesium chloride plant into one of the country's largest chlorine and caustic soda plants has been announced by Southern Alkali Corp.

This plant will produce more than 200 tons of liquid chlorine per day, and is expected to relieve some of the pressure on the liquid chlorine market which has made hand-to-mouth distribution of liquid chlorine necessary in order to insure a chlorine supply without interruption to the water supply and other essential fields.

408

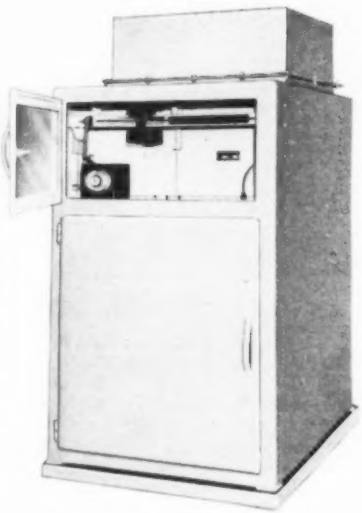
Novel Tracing Paper With Phantom Guide Lines

The Ogilvie Press, 691 Fulton Street, Brooklyn 17, N.Y. has developed a helpful new type of translucent cross section paper. The plainly visible phantom cross section ruling at quarter inch intervals on the sheet will not reproduce when making blueprints, black and white, or Ozalid reproductions. Made of 100% rag resin treated vellum, these papers will not become brittle or discolor with age. They are highly translucent and give excellent reproduction and possess a distinct advantage in that the engineer may draw in his own reference lines on graphs or sketches for reproduction. This is particularly advantageous in the preparation of graphs, charts or line-drawings for reproduction in magazines.

Samples are available. If you are interested, enter the above key number on a Reader Service Card and mail.

CONTROL YOUR FEEDING BY LOSS IN WEIGHT!

OMEGA CHEMICAL FEEDER



The OMEGA Loss-in-Weight Gravimetric Feeder delivers exactly by weight, the amount per hour required — shows at a glance the pounds of chemical fed. Feeding action is steady, precise and dependable, at rates from ¼ lb. per hr. upward, instantly adjustable by dial over a 100 to 1 range. With the Loss-In-Weight Feeder no catching or weighing of samples is ever necessary to determine the rate of feed. Almost any granular or powdered material, including lumps up to 1¼", can be fed with an accuracy of 99% or better. Automatic proportioning controls and alarm systems are available.

Write for Bulletin 20-23.

OMEGA PRODUCTS

Volumetric Feeders • Gravimetric Feeders • Solution Feeders • Lime Slaking Equipment • Bucket Elevators • Laboratory Stirrers

OMEGA MACHINE COMPANY

(Division of Builders Iron Foundry)

10 CODDING STREET • PROVIDENCE 1, R. I.

OMEGA

409

Zeolite Company Head Dies Company to Continue

Richard E. (Dick) Wagner, President of the Zeolite Chemical Company of New Jersey, died suddenly early in March. Dick, as he was familiarly known throughout the water works industry, was an enthusiastic flyer, having been an aviator before World War I. He continued to fly his own plane, a picture of which appeared in this magazine in the February issue.

The Zeolite Chemical Company will continue its operations. Dr. F. Tschirner, Chemist, who has been with the company since its inception, and Mr. C. E. Lord, Jr., Chemical Engineer, who has been connected with the company for the past sixteen years, will continue their responsibilities in the production of Zeolite.

Hardinge Appoints Agent for Southern Texas

The Hardinge Company, Inc., of York, Pa., has announced the appointment of the Chavigny Co., Houston, Texas, as sales agent for the Hardinge pulp and paper, water works, and sewage equipment in Southern Texas. This firm will handle all Hardinge equipment including clarifiers, automatic backwash rapid sand filters, digesters, etc.

T. H. Monaghan with Universal Concrete Pipe

Thomas H. Monaghan has been appointed assistant to the president and sales manager of the Universal Concrete Pipe Company, Columbus, Ohio.

A native of Chicago, and a graduate of Kenyon College at Gambier, Ohio, Monaghan started with Universal in the Pittsburgh office where he became sales manager and was later transferred to the headquarters at Columbus.

410

Pipe Joint Compound in Jumbo Sticks

The Lake Chemical Co., 607 No. Western Ave., Chicago, has announced the development of a larger diameter stick of pipe joint compound called the Jumbo Pipe-tite-Stik.

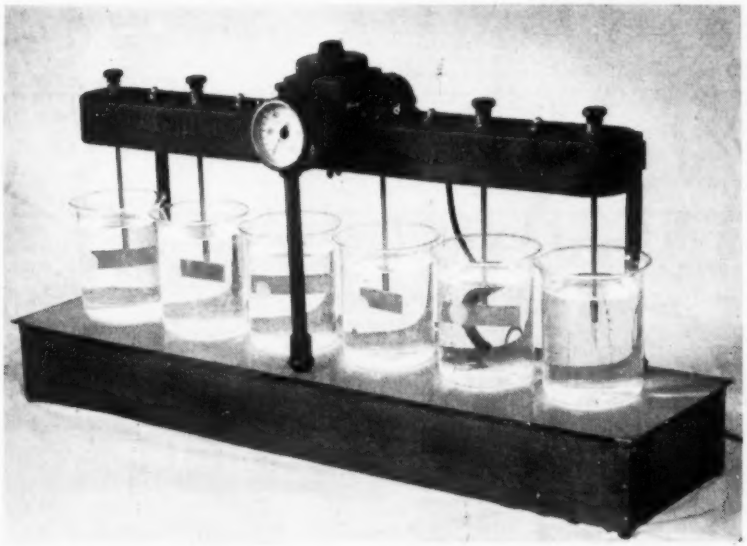
This pipe joint compound is applied by simply rubbing the stick over the pipe threads, where it spreads and fills the threads when turned. Pipetite-Stik withstands all sorts of chemicals, oils, water, and steam.

For further information mail a Reader Service card containing the above key number.

Rockwell Elects New Officers

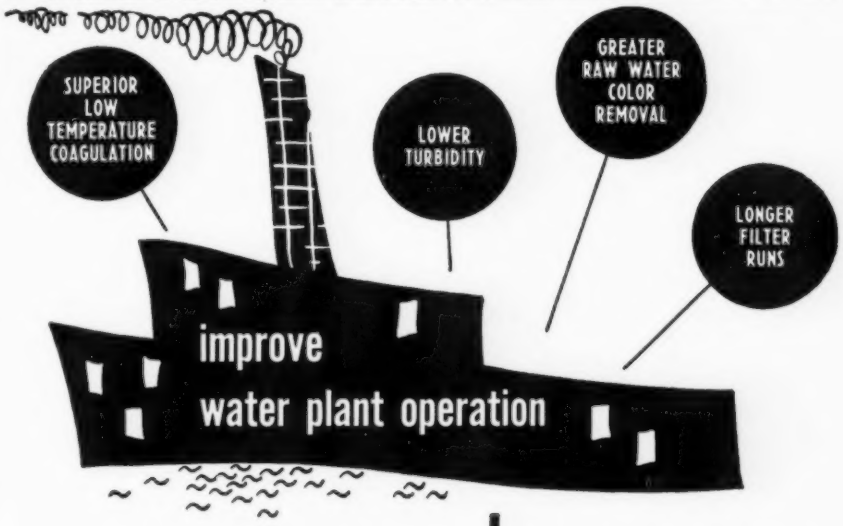
The Rockwell Manufacturing Company of Pittsburgh, Pa., has announced the election of four new officers at its annual organization meeting. J. E. Ashman was elected Vice President and Controller; E. W. Meyers as Secretary and M. J. Carl as Treasurer. Paul A. Wick and I. C. Rowe were elected Assistant Secretaries. All other officers of the Company were re-elected. W. F. Rockwell, Jr., is President of the Company.

THE *Phipps & Bird* LABORATORY MIXER - - -



IS the important piece of equipment in the Modern Water Works Laboratory. Now being used by leaders in water works research. Many superintendents and chemists have been able to reduce their chemical costs by the control of chemical dosages with this mixer. Write for literature.

PHIPPS & BIRD, INC.
RICHMOND, VA.
"SOUTHERN CENTER FOR LABORATORY APPARATUS AND CHEMICALS"



Want any of these for your plant? Let us send you a sample of "N" Silicate with directions for activating by any of the processes listed below. Jar tests prove how N-Sol forms large, tough, heavy floc to aid coagulation.

N-SOL PROCESSES FOR WATER COAGULATION

Developed and Patented by

PHILADELPHIA QUARTZ COMPANY

LICENSE AVAILABLE WITHOUT CHARGE • PHILADELPHIA 6

N-Sol-A* — N Silicate reacted with ammonium sulfate. Used where chloramine sterilization is desired.

N-Sol-B — N Silicate reacted with alum. (U. S. Pat. 2310009)

N-Sol-C* — N Silicate reacted with Chlorine. The residual hypochlorite retains the same oxidizing powers produced by normal chlorine sterilization.

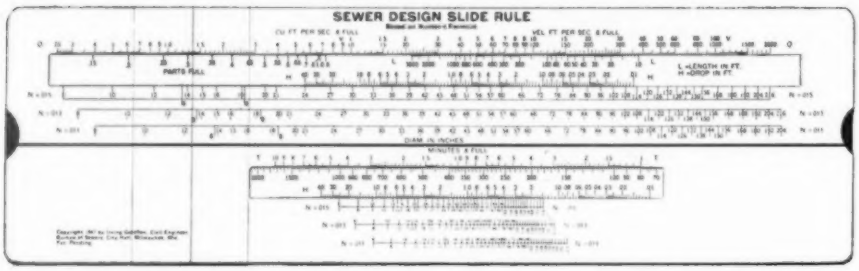
Baylis Sol — N Silicate reacted with sulphuric acid. (U. S. Pat. 2217466)

N-Sol-D — N Silicate reacted with sodium bicarbonate.

* Patent Applied For.

411

Sewer Design Slide Rule



A new sewer design slide rule has been developed by Irving Goldfein, Civil Engineer, 3052 N. 49th Street, Milwaukee 10, Wis. This slide rule is based on the Chezy formula and Kutter's formula. It has a wide range, is handy in size, simple to operate and rapid in the solution of sewer design problems. The rule is 3¾ in. by 13 in. long and consists of a rule and slide made of 6-ply, tough check, and a cellulose acetate runner with hairlines.

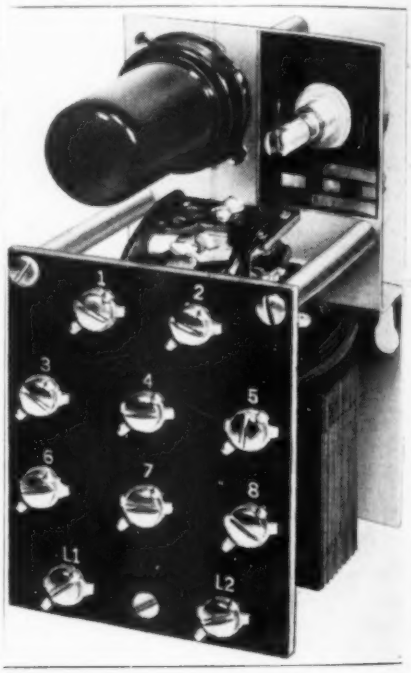
The back side of the device has the instructions for use, as well as a standard engineer's slide rule and the space for a name, professional card or advertisement.

For further information on this unique and handy tool for sewer design engineers, fill in a Reader Service card with above key number and mail.

412

Electronic Relay Control

R-S Products Corp., Philadelphia, Penna., has announced a new Leveltronic Relay. The new unit offers great sensitivity and wide range of applications. It is rugged and easily operated and includes a liquid level and interface control and alarm, pressure and flow control and alarm, temperature control and alarm, and high foam control and alarm.



Further information may be obtained through the use of a Reader Service Card.

413

Ferric Chloride Production

Liquid Ferric Chloride is now available from a Chicago producer, the Jordan Co., 51st Street and Merrimac Ave., Chicago, Ill. The Ferric Chloride may be obtained in tank car lots.

For further information, enter the above key number on a Reader Service Card and mail.

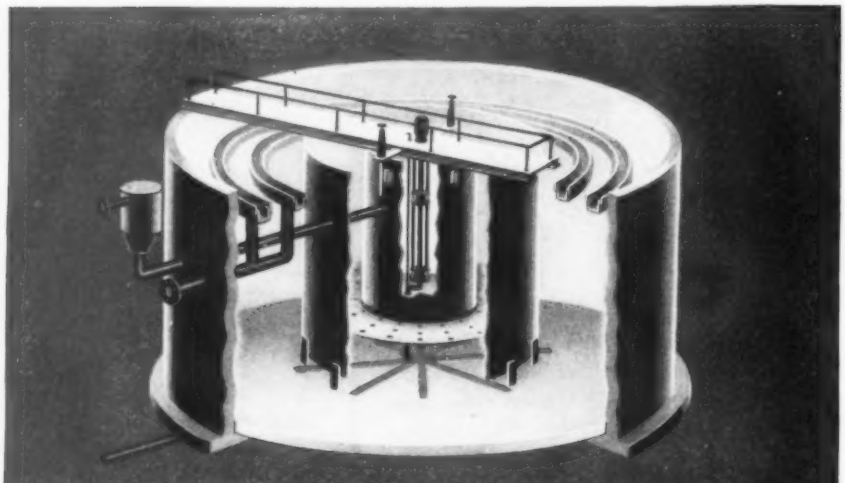
FOR WATER SOFTENING . . . TURBIDITY AND COLOR REMOVAL
. . . INDUSTRIAL WASTE TREATMENT . . .

The Clariflow

with independently-operated mixing, flocculation, stilling and sedimentation zones

A notable improvement over conventional short-retention water treatment units. Provides individual functioning of mixing, flocculation, stilling and sedimentation zones. • Its control over short-circuiting, control over each function and control of sedimentation through low weir rates mean improved effluent . . . higher filtration rates . . . longer filter runs . . . better industrial waste recovery and treatment.

Write for Bulletin 6S6



WALKER PROCESS EQUIPMENT INC.
EQUIPMENT • EQUIPMENT ENGINEERING
PROQUIP AURORA • ILLINOIS

FINE ENOUGH FOR THE

LARGEST Railroad SYSTEMS

Photo courtesy L. & N. Railway

HUNDREDS of Layne Well Water Systems are now serving the Nation's largest railroads. And only in these systems have maintenance engineers found the rugged quality, long life and high efficiency that creates such unmatched reliability. Layne is proud of this splendid tribute to their years of sincere craftsmanship.

But railroads are not alone in this acknowledgement of water system superiority. The same preference applies to thousands of installations for cities, factories, dairies, chemical plants, petroleum refineries and irrigation projects.

The buying of a Layne Well Water System automatically obtains the very best of everything;—Layne skill in modern well drilling;—the correct type of screen and its proper installation;—the proper pump for sustained economical service;—and the final tests to prove that the system is thoroughly efficient in its operation.

For literature on Layne Well Water Systems address Layne & Bowler, Inc., General Offices, Memphis 8, Tenn.

LAYNE WELL WATER Systems

AFFILIATED COMPANIES: Layne-Arkansas Co., Stuttgart, Ark. ★ Layne-Atlantic Co., Norfolk, Va. ★ Layne-Central Co., Memphis, Tenn. ★ Layne-Northern Co., Mishawaka, Ind. ★ Layne-Louisiana Co., Lake Charles, La. ★ Louisiana Well Co., Monroe, La. ★ Layne-New York Co., New York City ★ Layne-Northwest Co., Milwaukee, Wis. ★ Layne-Ohio Co., Columbus, Ohio ★ Layne-Pacific, Inc., Seattle, Wash. ★ Layne-Texas Co., Houston, Texas ★ Layne-Western Co., Kansas City, Mo. ★ Layne-Western Co. of Minn., Minneapolis, Minn. ★ International Water Supply Ltd., London, Ont., Can. ★ Layne-Hispano Americana, S. A., Mexico, D. F.

414

New pH Indicators

LaMotte Chemical Products Co., Towson, Baltimore 4, Md., has developed new water soluble pH indicators. These are brilliant crystals of pure dyes, instantly soluble in distilled water, and require no mixing in mortar, no filtering, and no waiting. For further information, use a Reader Service card and the above key number.

Diamond Alkali Moves Offices

The executive offices of the Diamond Alkali Co. have been moved from Pittsburgh to the Union Commerce Bldg. in Cleveland. The company has had its offices in Pittsburgh since 1910. The move was made because of the national expansion of operations by the company, according to President Raymond F. Evans. The new location is a more central spot for integrated control of the company.

Galvanizing Motion Picture Available

A 16 mm. motion picture sound film depicting zinc coating by the hot dip galvanizing method is available. It is the first such visual medium used to acquaint an audience with this process of protective coating.

Titled "Must It Rust?" the film showing time is 25 minutes. It shows the steel base metal actually rusting as viewed through a microscope, cleaning the metal, and galvanizing.

All requests for showings should be addressed to Mr. Stuart J. Swensson, Secy.-Treas., American Hot Dip Galvanizers' Association, Inc., 1611 First National Bank Bldg., Pittsburgh 22, Pa.

415

Glass Ware Cleaner

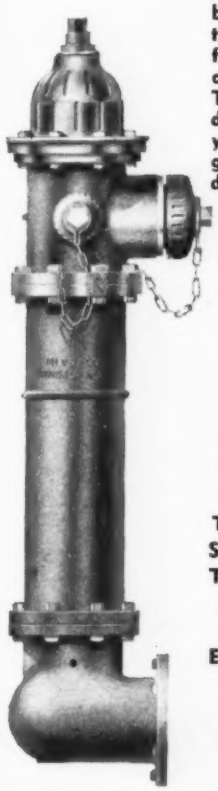
The Burrell Technical Supply Co., 1942 Fifth Ave., Pittsburgh 19, Pa., offers a new wetting agent known as Alconox, for cleaning glass ware, machine parts, metal, cloth, walls, porcelain and pipettes. This soapless cleaner leaves no film and no streaks, but works rapidly to lift dirt, grime and grease. It is gentle on the hands.

For further information enter the above key number on a Reader Service card, and mail.

M H

HYDRANTS and VALVES

Expert workmanship and highest quality materials make M & H products a good investment always. Many years of use in all sections of the country has proved their ruggedness and dependability.



M & H furnishes both regular type A.W.W.A. fire hydrants and special Traffic Model designed to yield at the ground line under impact.

Write for Catalog No. 34

M & H PRODUCTS
Fire Hydrants
Gate Valves
Tapping Valves
Special Castings
Tapping Sleeves
Check Valves
Floor Stands
Extension Stems
Mud Valves
Flap Valves
Sludge Shoes

M & H VALVE AND FITTINGS COMPANY
ANNISTON, ALABAMA

Mathieson's Packets of Emergency Health Measures

The Mathieson Alkali Works of New York has prepared a packet of material designed to advise the public of necessary emergency health measures. The packet contains a set of releases for use in newspapers before, during and after a flood, a similar set of releases for radio broadcasts, a bulletin for posting in public places, and a cartoon feature for newspaper use. These packets are distributed to State Health Offices in areas likely to be affected by floods.

Information in the packet covers recommendations for typhoid inoculation, treatment of water for drinking and cooking purposes, handling of contaminated food and containers, disinfection of wells, and clean-up of buildings. Also included in the packet is a list of distributors of sanitation chemicals in the affected area and night and day telephone numbers of individuals prepared to assist when emergency sanitation is necessary. The U. S. Public Health Service and Red Cross Officials have also been supplied with this list.

M. E. Barton to Represent Philadelphia Quartz in Mid-West

The Philadelphia Quartz Company has announced the appointment of Miller E. Barton to its sales staff. Mr. Barton assumes duties as a representative in the Midwest territory, residing in Independence, Mo. Mr. Barton's territory will include the states of Kansas, Missouri, Nebraska and Iowa, and he will assist Clement F. Lovely, senior representative in that area.

For the Utmost in Turbine Pump UTILITY AND CAPACITY

PEERLESS Vertical Turbine PUMPS

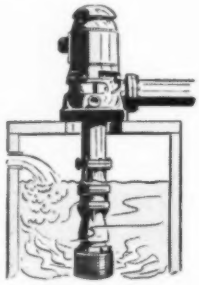
PEERLESS Builds a Pump to Suit Your Service—Capacities: 15 to 30,000 G.P.M.

PEERLESS VERTICAL TURBINE PUMPS FOR DEEP WELL SERVICE

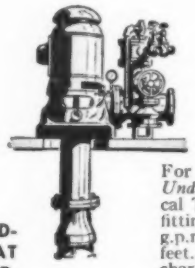
For development of a reliable, permanent water supply, install these deep well pumps with the type of drive most suitable to your application. You have a choice of electric, right angle gear drive, or belted (V- or flat) heads as well as the option of oil or water lubrication. Capacities from 15 to 30,000 g.p.m. are obtained from depths of 1000 feet or more. *Fully described in Peerless Bulletin B-141-1.*

PEERLESS CLOSE-COUPLED TYPE VERTICAL TURBINE PUMPS

For pumping from short settings such as pits, sumps and surface water sources, such as lakes, rivers and ponds, get the full utility and capacity of a deep well pump with a close-coupled Peerless Vertical Turbine Pump. Makes an extremely compact installation. Uses include cooling tower service, boosting, recirculation and dewatering. Capacities: up to 30,000 g.p.m. *Fully described in Bulletin B-159.*

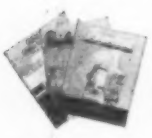


PLAN YOUR PUMP REQUIREMENTS WITH PEERLESS— PEOPLE WHO CARE



PEERLESS VERTICAL TURBINE APPROVED FIRE PUMPS...

For adequate fire protection, the *Underwriters' approved* Peerless Vertical Turbine Fire Pump, with proper fittings, produces from 500 to 2000 g.p.m. against heads from 240 to 285 feet. Built for instant operation. Discharge piping is simplified. Choice of drives and oil or water lubrication. *Fully described in Bulletin B-152.*



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PEERLESS PUMP DIVISION

Food Machinery Corporation

Factories: Los Angeles 31, Calif.; Quincy, Ill.; Indianapolis, Ind. *District Offices:* New York 5, 37 Wall Street; Chicago 40, 4554 North Broadway; Atlanta Office: Rutland Building, Decatur, Georgia; Dallas 1, Texas; Fresno, Calif.; Los Angeles 31, Calif.

fmc PEERLESS PUMP

Dearborn News

The Dearborn Chemical Co. of Chicago, Ill., has elected E. J. McMahon, Production Manager, to the position of Director and Vice-President. Mr. McMahon succeeds John W. Brashears who has been with the Dearborn Company for 52 years.

E. J. McMahon

F. A. Jones

The company has also appointed Frank A. Jones, formerly associated with the Terminal Railway Company of St. Louis, to the position of Chief Engineer. He succeeds Mr. E. A. Goodnow, who has relinquished his position, due to ill health.

The company has also appointed Gordon E. MacLean as representative of the Sales Department, to be located in the Indianapolis District Office. Mr. MacLean has spent seven years in the laboratory of the company, developing corrosion inhibitors and doing research work on cleaners.

Glass & Assoc. to Represent Ralph B. Carter Co.

Announcement has been made that Harry J. Glass and Associates of Colorado Springs, Colo., will henceforth be sales representatives for the Ralph B. Carter Co. of Hackensack, N.J., handling Carter mechanical equipment for water and sewage works in the states of Colorado and Wyoming.

FOR

pH Control and WATER TESTS

A study of our 28-page Bulletin No. 602, containing the complete line of Hellige apparatus for pH control and water analysis, will enable you to form an intelligent opinion as to what is best suited for extending your present equipment or for an initial installation.

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INCORPORATED
3718 NORTHERN BLVD. LONG ISLAND CITY 1, N.Y.
HEADQUARTERS FOR COLORIMETRIC APPARATUS

CLIMAX
High Pressure Test Plugs
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For Bell & Spigot Cast Iron Pipe. Simple operation. All sizes carried in stock.

These plugs valuable for testing steel pipe lines. A complete set of both types on your repair trucks will prove a good investment.

Write today for complete information

THE
RICHMOND
FOUNDRY & MANUFACTURING CO.
INC.
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416
Fire Proof Finish

Monsanto Chemical Co. of St. Louis, Mo., has developed a fireproof finish, known as Skylac. This material, which is used on airplanes, gives a smooth finish, is waterproof, and will not catch fire, even under the influence of flames. Skylac may be used on either interiors or exteriors, and is not affected by sun, ice or rain.

For further information, enter the above key number on a Reader Service Card and mail.

LITERATURE AND CATALOGS

417
Pipe Location

Fisher Research Laboratory, Inc., of Palo Alto, Calif., has a four page folder entitled **"M-Scope News** for Utilities, Municipalities and Oil Companies". This folder shows the M-Scope leak locator, and how to use it. Also shown is the new Midget Combination Leak Detector-Pipe Finder.

You may obtain a copy of this booklet by entering the above key number on a Reader Service Card.

418
Elevated Tanks

The Chicago Bridge & Iron Co., 332 S. Michigan Ave., Chicago, Ill., has published one of the most unusual pieces of printed matter that they have issued in the past 25 years. The bulletin, which contains 22 pages, is titled **Horton Ellipsoidal Bottom Elevated Steel Tanks of Welded Construction.**

The bulletin shows seven different types of tanks printed in attractive color, with each tank and its surrounding background done as an individual artist's water color.

In addition to these attractive pictures, the book also contains information on the construction of the tanks, purposes of the tanks, effect on reducing pumping costs, increases in capacity of the system, where to locate the tanks, the relation of elevated tanks to fire flow requirements, and many other items of interest and information of value to waterworks superintendents and managers.

If you desire a copy of this very interesting bulletin, just enter the above key number on a Reader Service Card and mail to this magazine.

FISHER M-SCOPE
ELECTRONIC
LEAK DETECTOR

'HAN-DEE' MIDGET DETECTOR LOCATES LEAKS PRECISELY

Why tolerate costly leaks in oil, water, sewage, and pressure lines . . . or waste enormous sums in blind exploratory excavating to find them? The M-Scope "Han-Dee" Midget Leak Finder, an inexpensive electronic instrument, is so sensitive that it can locate the exact trouble with pin-point accuracy . . . even through concrete.

$67.50
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Write today for **FREE** *16 Page Booklet*

Fisher Research Laboratory, Inc.
PALO ALTO
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419

Gas Safety Improvement

The Pacific Flush-Tank Co., 4241 Ravenswood Ave., Chicago 13, Ill., has a new bulletin which "takes you inside improved P.F.T. safety equipment".

Bulletin 221, titled **Gas Safety Equipment,** shows flame traps, pressure relief with waste gas flame trap, pressure relief waste gas burners, gas pressure indicating gauge, low pressure check valve, drip traps, accumulators for condensate and sediment, etc.

Any sewage treatment works where gas is produced in sludge digestion tanks should have copy of Bulletin 221. To obtain one, enter the above key number on a Reader Service Card and mail.

420

Sewage Pumps

Peerless Pump Division of Food Machinery Corp., Los Angeles 31, Calif., has issued Bulletin No. B-154 on **Peerless Non-Clog Sewage Pumps.** This bulletin covers the vertical dry basin and horizontal types NCV-NCH sewage pumps.

The bulletin describes the non-clogging action of these pumps, shows pictures of the different types, gives sizes, and other information of interest.

To obtain a copy of Bulletin No. B-154, enter the above key number on a Reader Service Card.

421

Sewer Cleaning Equipment

Flexible Sewer Rod Equipment Co., 9059 Venice Blvd., Los Angeles 34, Calif., has published a 36 page catalog on its equipment for cleaning sewers. The catalog contains pictures and directions for operating "Flexibles" and pictures of all types of equipment, including the cork screws, spear head, double cork screw, pick-up tool, etc., etc. The Bulletin also contains pictures of roots removed from various sewers and illustrations of the rod and reel stand, one-man ratchet, power-drive, and sewer brushes.

There is a section on sand, gravel, mud, and rocks, another on water power cleaning, and illustrations of lateral locators and rubber pipe plugs, as well as the "carry-all" trailer, power bucket machine and the winch power drive. The book concludes with a section on the "Flexible" Hi-Way Culvert Scraper, and a list of municipalities which have used "Flexibles" for sewer cleaning.

If you are interested in sewer cleaning, you will want this book in your office. To obtain it just fill in a Reader Service Card and mail.

422

Lightweight Pumps

The Gorman-Rupp Co., Mansfield, Ohio, has published a four-page folder on Gorman-Rupp's **Self-Priming Centrifugal Lightweight Pumps.**

This is the new pump recently described in our Equipment News item in three sizes—the Midget, a 1½ in. pump, the Hawk, a 2 in. pump, and the Eagle, a 3 in. pump.

For a copy of this folder describing these pumps and their application, use a Reader Service Card on which is entered the above key number.

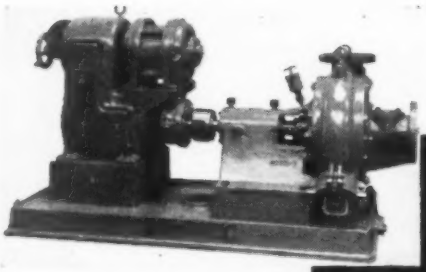
423

Liquid Level Controls

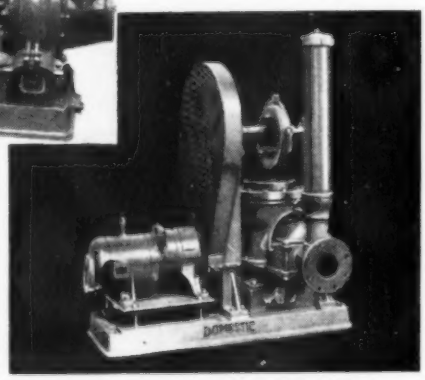
The B/W Cotnroller Corp., Birmingham, Mich., has issued a folder, M-4, on **Liquid Level Controls.** These are the all electric floatless controls manufactured by the B/W company.

A copy of this folder is available. If you desire one, enter the above key number on a Reader Service Card.

PUMPS *that can "take it" and "dish it out!"*



PLUNGER
***Sludge* PUMPS**



SEWAGE PUMPS

Above—4 in. Horizontal Sewage Pump with Varispeed Motor. Handles sewage with efficiency and economy. Also available, Vertical unit for varying conditions of service.

Above—EPS Simplex Type—Accessible and dependable. Also available—Fixed and adjustable stroke—Simplex, Duplex and Triplex Units.

Dealers Note

A few territories are open for responsible dealers. Communications will be held in strict confidence.

Domestic has for 44 years produced an outstanding line of multi-purpose pumps. Every one is engineered for quality performance. In addition, special problems get prompt attention from our engineering department. Write Dept. W for FREE catalog.

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DOMESTIC ENGINE & PUMP COMPANY

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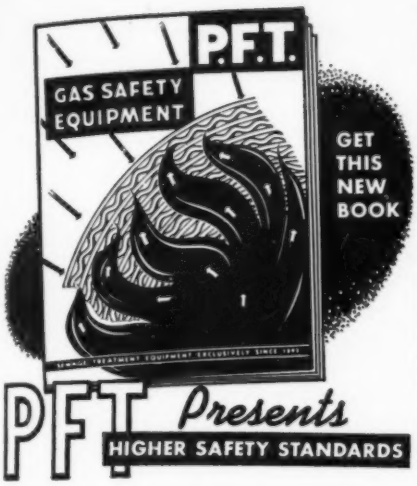

PFT Presents HIGHER SAFETY STANDARDS

New Bulletin 221 describes the recent improvements in P.F.T. Gas Safety Equipment; for better protection for boiler rooms and other installations, and longer service life for the equipments. All units are illustrated with detailed drawings. Specifications, typical gas piping arrangements and charts for selecting sizes are included.

P.F.T. PACIFIC FLUSH-TANK CO.
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NEW YORK • LOS ANGELES
SAN FRANCISCO • DENVER
CHARLOTTE, N. C. • TORONTO

424

Chemical Feeders

Omega Machine Co., 9 Codding St., Providence 1, R. I., has available two bulletins on chemical feeders. The first on Universal Feeders is a four page folder describing the design and operation of the Universal feeders which in three sizes will cover a feeding range of from one lb. to 5,000 lbs. per hour.

The companion bulletin D1-10-20 is a four page folder which presents pictures and brief descriptions of eight different types of Omega feeders, including the bell-type gravimetric feeder, universal feeder, loss-in-weight gravimetric feeder, precision solution feeder, groove disc feeder, rotodip solution feeder, rotolock feeder, and dust collectors. Each of these feeders has a bulletin about it and this folder gives the bulletin number for each feeder.

If you desire information on dry feeders, enter the above key number on a Reader Service Card and mail.

425

Cloroben

The Cloroben Corp., Jersey City 2, N. J., has issued an 8-page pamphlet, titled, **"The Cloroben Manual** of Sewage and Sewer Maintenance Problems". This manual tells what Cloroben is, and explains its development for use as an up-sewer treatment chemical. The bulletin explains what Cloroben treatment will do to remove slime from sewer walls, reduce bacteria and prevent hydrogen sulfide production. Also explained is a method for establishing up-sewer Cloroben treatment, how to use Cloroben in the disposal plant and its use on screenings, grit, skimmings and sludge beds to prevent odors.

Copies of the Cloroben Manual are available for distribution. To obtain one, mail a Reader Service Card containing the above key number.

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THE LEADITE COMPANY
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No Caulking

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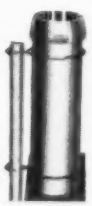
Specialized Industrial Cleaning
MATERIALS • METHODS • SERVICE



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RAIN GAGE

A versatile, low cost, precision instrument for measuring precipitation in all forms. Using interchangeable parts many different assemblies can be made up for rain or snow, heated gages, remote indicating and accumulating gages. Total cost of parts for basic Rain Gage only $4.75.

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Ashburnham Massachusetts

HAVE YOU TRIED
stainless steel
MEMPHIS SLOT INSERTS
with new
bronze thrust rollers?

PATENTED

SAVE MONEY, LABOR, MEASURING CHAMBERS with this simple device, easily installed, made to fit water meters varying in size from 5/8" to 2". By lining ragged roller slots, the Memphis Slot Insert adds years to the life of measuring chambers and provides remarkable accuracy in water registration.
For free 5/8" samples, send the coupon below to the

METER SPECIALTY CO.
Office: Sterick Bldg. Factory: 1332 N. Seventh St.
MEMPHIS, TENNESSEE

Please send me free samples of Memphis Slot Inserts to fit 5/8" water meters of the following makes:

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Your Name..... Title.....

Company.....

Address.....

.....

426

Ranney Water Collectors

Ranney Water Collector Corp. of New York, 33 West 42nd St., New York 18, N. Y., issued a reprint titled "How Ranney Type Collectors Solved the Water Problem for Hartford Rayon Corporation", reprinted from the Hartford Rayon Reporter. The reprint contains a list of advantages of Ranney Water Collectors, as well as the story on the Hartford Rayon Corp. supply.

Copy of this reprint is available. If desired, enter above key number on a Reader Service Card.

427

Lubricated Valves

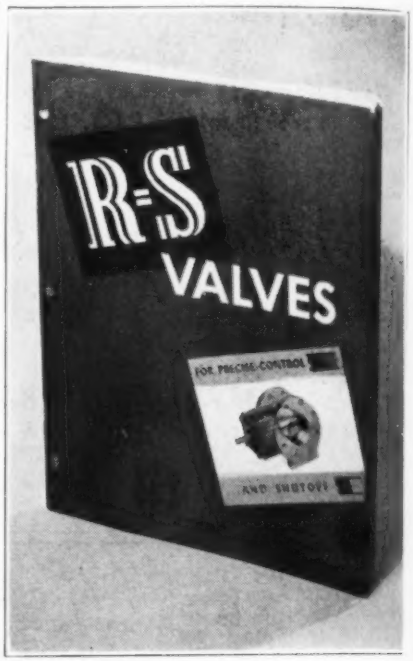
The Nordstrom Valve Division of the Rockwell Manufacturing Company, 400 N. Lexington Ave., Pittsburgh 8, Pa., has issued Bulletin V-200 on **Nordstrom Lubricated Valves.** This 82 page catalog contains a complete presentation of all types of these semi-steel, Nordco Steel, Standard Type, Hypreseal Type, Straightway and Multiport, Air and Service Cocks in the plug valve lines as well as all accessories.

A copy of Bulletin V-200 is available. If you desire one enter the above key number on a reader service card and mail to the Reader Service Dept. of this magazine.

428

Valves

R-S Products Corporation, Wayne Junction, Philadelphia 44, Pa., has just released a new illustrated valve catalog, titled **"R-S Valves".** This 52 page volume is fully illustrated, and well indexed. It gives a comprehensive coverage of disc type valves and includes more than 70 specially made photographs of various valves, contains tables keyed to drawings, showing dimensions of valves, and shows 11 basic types of R-S butterfly valves, including detailed elevations which show manual controls, etc. A special section is devoted to prime movers, and shows a score or more with dimensional data recommended for use with the valves described.



This catalog No. 17 is available upon request. To obtain one, enter the above key number on a Reader Service Card and mail to Reader Service Dept. of *Water & Sewage Works.*

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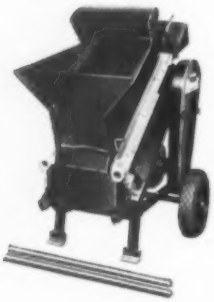
SHOW A LOWER YEARLY OPERATING COST . . . .

with a Royer Sludge Disintegrator

Obtain revenue which will make your yearly net operating costs a real credit to you. Convert your sludge into a marketable fertilizer with the Royer Sludge Disintegrator.

It costs less to Royerate sludge than it does to burn or bury it. Just shovel sludge cake from drying beds into hopper. The rugged, inexpensive machine discharges an effective, ready-to-use fertilizer to bring in money that will offset expenses. Send for "Sewage Sludge Utilization Catalog"—it shows how plants, large and small, are reaping revenue from sludge beds.



Model NSC-2, capacity 5 to 8 cu. yds. per hr. One of 12 models: electric motor gasoline engine and belt-to-tractor driven.

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PIONEERS IN THE SALVAGE OF SEWAGE SLUDGE

ROYER FOUNDRY & MACHINE CO.

170 PRINGLE ST., KINGSTON, PA.

The following listed new Class B, cast iron water main is in stock for immediate shipment; subject to prior sale:

4	pc.	—24"x18' Class B, C.I. Simplex Lock Joint Pipe
2	"	—24"x18' Class B, C.I. B&S pipe
1	"	—24"x12' ditto
5	"	—20"x18'
1	"	—20"x12'
2	"	—18"x12'
6	"	—16"x18'
1	"	—16"x10'
1	"	—16"x 9'
10	"	—14"x18'
4	"	—14"x12'
9	"	—12"x18'
16	"	—10"x18'
3	"	—10"x12'
1	"	— 6"x18'
1	"	—10"x12'—5" C.I. flanged, faced & drilled

DAVE GORDON & COMPANY

P.O. Box 5116 Tampa, Fla.

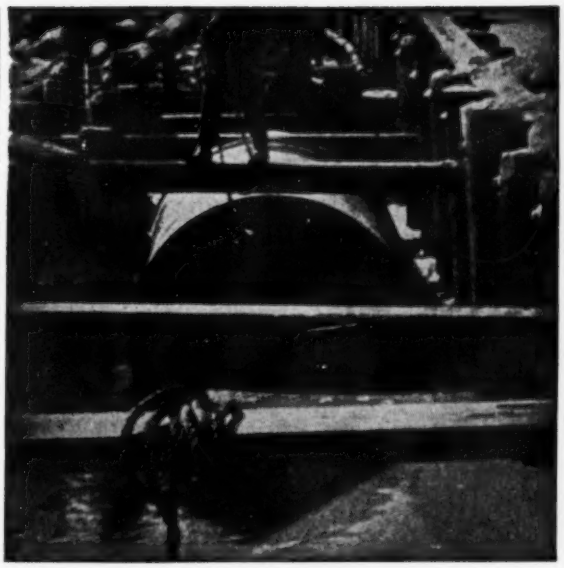
429

Gate Valve Operation

Payne Dean & Company of Madison, Conn., has issued a 1948 edition of **"Gate Valve Operation".** This bulletin shows the new Gatender, the portable Powerench and the truck mounted gate valve operator as several units in this field of gate valve operation. Shown in detail is the design of the power takeoff, the operating head and the portable units.

The Dean Gatender can be a self-propulsion model to which can be added a snow plow for cleaning walks, or it can operate a gang mower. A Powerench is the latest development in small gate operators and utility power units. It is completely self contained with its own power unit. The Gatender has a variety of uses and can operate either vertically or horizontally. All of these uses for the Gate Valve Operators are shown in the bulletin.

Any water works superintendent or operator who has problems of large valve operations will certainly be interested in this bulletin. To obtain a copy, enter the above key number on one of the Reader Service Cards in this magazine and mail to our New York office.



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- Featherweight - only 11 lbs. complete.
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- Operating simplicity. A one-man instrument.
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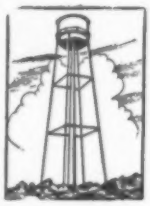

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ESTIMATES NO OBLIGATION

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430

Jointing Compound

Northrop & Company, Inc., Spring Valley, N. Y., have just issued a 16 page catalog titled **Machine Blended Bond-O.** Bond-O is a self-caulking compound for jointing water mains, and this catalog describes the material, explains the selection of the basic minerals, tells how the minerals are controlled, and discusses the science of machine-blending. Bond-O which may be obtained in powder form in packages or in ingots, contains sulphur, carbon, iron and silicon, which are machine-blended and homogenized to obtain thorough and complete mixing and distribution of the various chemicals.

The bulletin also discusses the advantages of the powder versus ingots, the sealing action of Bond-O, and how to use the material on wet joints and pipe lines under water, lines without pressure in split sleeves or on bridges. There is also a discussion of the sulphur bacteria and their action on sulphur joint compounds, and the fact that germicides can be used in these materials to prevent bacterial action.

A copy of this bulletin is available, all you need to do is to enter the above key number on a Reader Service Card and mail it.

FOR SALE: 1—3500 KW 3 Phase, 60 Cycle, 2300 Volts, GE Turbo Generator; Complete factory overhaul. 1—Curtis 200 lb. pressure, 100 degrees superheat, steam, condensing, 3600 RPM Turbine. 1—Braun Surface Condenser (New) 135 HP Type FTS size 76-192 Low Level Jet, with 56" inlet nozzle and single bellows type expansion joint. Lucey Products Corporation—Tulsa, Oklahoma.

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*(Director of Laboratories, Dept. of Water Supply, City of New York)

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Of Iron . . . Taste . . . Odors . . . . . Colors . . . Suspended Matter . . .

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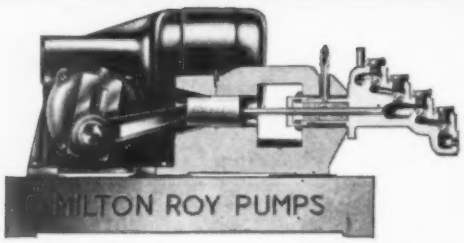

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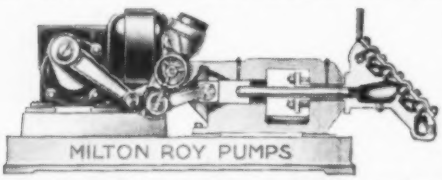

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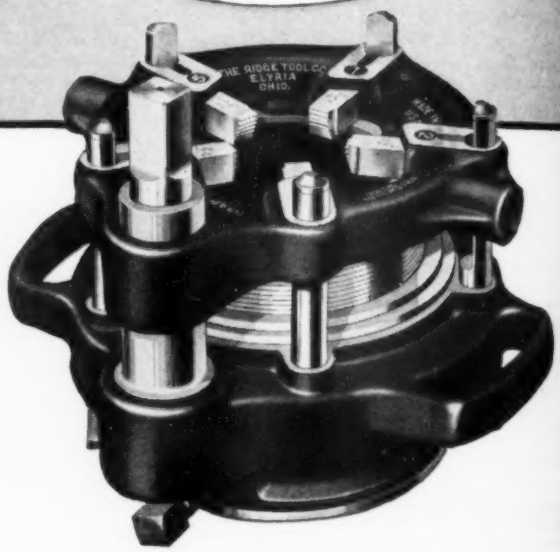

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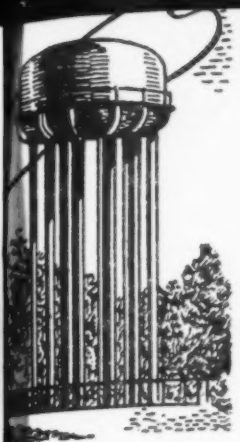

PART 1

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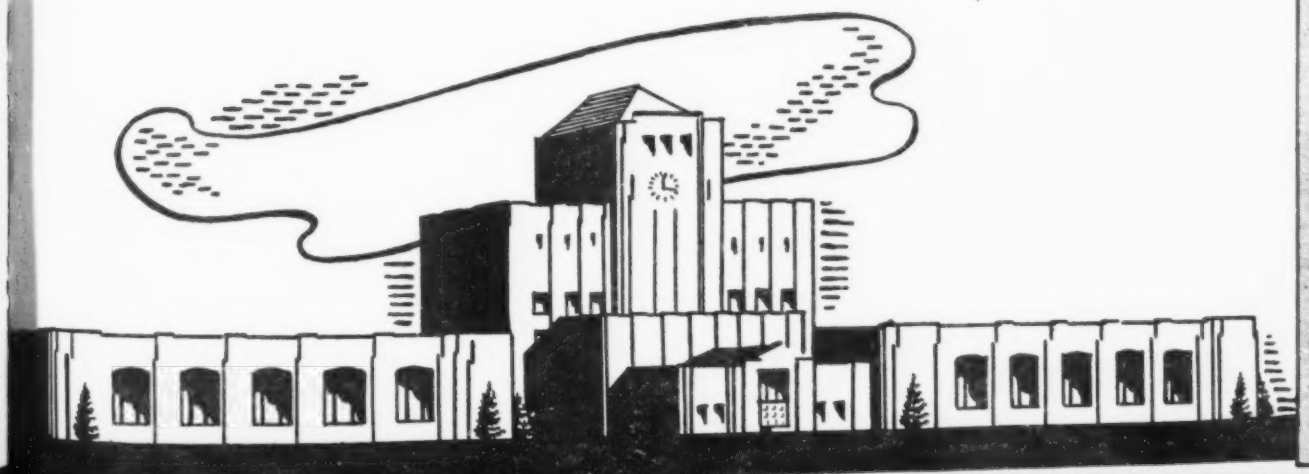

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Foreword

We take especial pride, as publishers, in presenting this volume—

The 1948 Reference and Data Number of

Water & Sewage Works

Prepared by the Editor and Dr. George E. Symons, as Managing Editor, this section presents the equivalent of a data handbook for designing engineers, managers, and operators of water and sewage works.

The information contained herein has been carefully selected from past issues of *Water Works and Sewerage* (now *Water & Sewage Works*) and other sources named in the text. In addition, considerable effort has been made to present the most economical and modern methods, and the latest tables and charts useful to water and sewage works operators and designers.

Even though supplied to subscribers at no charge beyond the regular subscription fee, the same conscientious work has been put into the preparation of this Reference Section as would be expected in a book devoted exclusively to the same type of material and published separately.

* * *

Note

In the past we have repeated useful basic data, tables, graphs, etc.—from year to year, chiefly for the benefit of new subscribers; certain other matter was repeated in alternate years; and new material was, of course, added. Thus the subscriber who kept two consecutive Reference and Data issues on file had all of the most important reference material published by Water Works and Sewerage (now Water & Sewage Works).

None of the material contained in the 1946 and 1947 issues is repeated in this issue. Instead, the new material is being augmented with certain basic material selected from the 1945 and earlier issues, with the intent of best serving all of our readers—especially the newer readers.

In order to have a complete Reference and Data file it will be necessary for the readers to have available not only the current issue but also issues of at least the two years preceding this issue. Several articles published since 1941 have not been repeated within the last three years. Therefore for the convenience of the user a consolidated eight year index is being incorporated in this issue.

L. H. Enslow

Editor.

The following pages contain names and addresses of manufacturers of water, sewage, and industrial waste treatment equipment and supplies. It is arranged for your convenience.–Make use of it.

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FOR THE INDEX TO THE REFERENCE AND DATA MATERIAL, AND THE INDEX TO ADVERTISERS, SEE THE REAR OF THIS SECTION

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Jeffrey Mfg. Co., Columbus, Ohio.
Lakeside Engrg. Corp., 222 W. Adams St., Chicago, Ill.
Link-Belt Co., 301 W. Pershing Rd., Chicago 9, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Milton Roy Co., Chestnut Hill, Philadelphia 18, Pa.
Omega Machine Co., 9 Codding St., Providence 1, R. I.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Roberts Filter Mfg. Co., Darby, Penn.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
Walker Process Equip., Inc., Aurora, Ill.
Worthington Pump & Machinery Corp., Harrison, New Jersey.

BOILERS, GAS
American Radiator & Standard Sanitary Corp., Bessemer Bldg., Pittsburgh 30, Pa.

BOOKS, TECHNICAL
John Wiley & Sons, Inc., 440 4th Ave., New York 16, N. Y.

BOXES, VALVE AND METER
(See Valve Boxes; Meter Boxes)

BRASS GOODS
(See also Pipe, Brass)
American Brass Co., Waterbury, Conn.
Hays Mfg. Co., Erie, Pa.

Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

BREECHINGS, STEEL
Chicago Bridge & Iron Company, 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

BUFFER SOLUTIONS
Heilige, Inc., 3718 Northern Blvd., Long Island City, N. Y.
La Motte Chemical Co., Towson 4, Md.
W. A. Taylor & Co., 7300 York Rd., Baltimore, Md.

BUTTERFLY VALVES
(See Valves, Butterfly)

CABLE
General Electric Corp., Schenectady, N. Y.
Westinghouse Electric Corp., Pittsburgh, Pa.

CALCIUM HYPOCHLORITE
Innis, Speiden & Co., 117 Liberty St., New York City.
Mathieson Alkali Wks., 60 E. 42nd St., New York City.
Penn. Salt Mfg. Co., Widener Bldg., Philadelphia 7, Pa.
Pittsburgh Plate Glass Co., Columbia Chemical Division, Grant Bldg., Pittsburgh 19, Pa.
E. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.
Wood and Preuit, 1740 K St., N.W., Washington, D. C.

CALGON
(See Corrosion Suppression and Water Stabilization)

CAULKING MACHINES & TOOLS
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., 50 Church St., New York 7, N. Y.

CARBON-ACTIVATED
(See Activated Carbon)

CARBONATORS (Recarbonation).
Carborundum Co., Perth Amboy, N. J.
Dorr Co., 570 Lexington Ave., New York City.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
F. B. Leopold Co., Inc., 422 First Ave., Pittsburgh, Pa.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Vogt Mfg. Co., Louisville, Ky.
Walker Process Equipment Co., Aurora, Ill.

CASTINGS, IRON, STEEL, ALLOY STEEL, ETC.
Central Foundry Co., 386 Fourth Ave., New York, N. Y.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Climax Engineering Co., Clinton, Iowa.
Elmco Corp., P. O. Box 300, Salt Lake City, Utah.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hardinge Mfg. Co., 240 Arch St., York, Pa.
James B. Clow Co., 201 N. Tallman Ave., Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
M & H Valve & Fittings Co., Anniston, Ala.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.

CAST IRON PIPE
(See Pipe)

CATHODIC PROTECTIVE SYSTEMS
Electro Rust-Proofing Co., Belleville, N. J.
Federal Telephone and Radio Corp., Newark 1, N. J.
Johnston & Jennings Co., Rusta Restor Div., 877 Addison Rd., Cleveland, Ohio.
Westinghouse Electric & Mfg. Co., P. O. Box 868, Pittsburgh 30, Pa.

CAUSTIC SODA
Diamond Alkali Co., Pittsburgh, Pa.
Hooker Electrochemical Co., Buffalo Ave. and Ward St., Niagara Falls, N. Y.
Innis-Speiden & Co., 117 Liberty St., New York City.
Mathieson Alkali Wks., 60 E. 42nd St., New York City.
Niagara Alkali Co., 60 E. 42nd St., New York City.
Penn. Salt Mfg. Co., Widener Bldg., Philadelphia 7, Pa.
Pittsburgh Plate Glass Co., Columbia Chemical Division, 30 Rockefeller Plaza, New York City.
E. M. Sergeant Pulp & Chemical Co., Inc., Empire State Bldg., New York 1, N. Y.
Solvay Sales Corp., 40 Rector St., New York City.

CEMENT ASBESTOS PIPE
Johns-Manville, 22 E. 40th St., New York City.

CEMENT GUN
Cement Gun Co., Allentown, Pa.

CEMENT PIPE
(See Pipe, Concrete)

CEMENT LINED PIPE
(See Pipe)

CEMENT LINING OF PIPE
Centriline Corp., 140 Cedar St., New York City.
Pre Load Industries, 420 Lexington Ave., New York 17, N. Y.

CEMENTS, ACID PROOF
Atlas Mineral Prod. Co., Mertztown, Pa.
Northrup & Co., Inc., 50 Church St., New York 7, N. Y.
Pennsylvania Salt Mfg. Co., 1000 Widener Bldg., Philadelphia 7, Pa.

CEMENTS, REFRACTORY
Carborundum Co., Perth Amboy, N. J.
Johns-Manville, 22 E. 40th St., New York City.

CEMENTS, SEWER PIPE
Atlas Mineral Prod. Co., Mertztown, Pa.
Hydraulic Development Corp., 50 Church St., New York City.
Pennsylvania Salt Mfg. Co., 1000 Widener Bldg., Philadelphia 7, Pa.

CEMENTS, WATER MAINS
Atlas Mineral Prod. Co., Mertztown, Pa.
Hydraulic Development Corp., 50 Church St., New York City.
Leadite Co., Girard Trust Bldg., Philadelphia, Pa.
Michael Hayman & Co., 870 E. Ferry St., Buffalo, N. Y.
Northrup & Co., Inc., 50 Church St., New York 7, N. Y.

CHAINS (Elevating, Conveying & Power Transmission)
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.

CHEMICAL, CONVEYING EQUIPMENT
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dracco Corp., Cleveland, Ohio.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.

CHEMICAL FEEDING & PROPORTIONING EQUIPMENT
American Water Softener Co., Philadelphia 33, Pa.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Everson Mfg. Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Lakeside Engrg. Corp., 222 W. Adams St., Chicago, Ill.
F. B. Leopold, 422 First Ave., Pittsburgh, Pa.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Milton Roy Co., Chestnut Hill, Philadelphia 18, Pa.
Omega Machine Co., 9 Codding St., Providence 1, R. I.
Permutit Co., 330 W. 42nd St., New York City.
Phipps & Bird, Inc., Sixth & Byrd, Richmond, Va.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Roberts Filter Mfg. Co., Darby, Pa.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

CHLORINATORS
(See also Hypochlorinators)
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Lakeside Engrg. Corp., 222 W. Adams St., Chicago, Ill.
Omega Machine Co., 9 Codding St., Providence 1, R. I.
Phipps & Bird, Inc., Sixth & Byrd, St., Richmond, Va.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Wallace & Tiernan Co., Inc., Newark, N. J.

CHLORINE DIOXIDE
(See Sodium Chlorite)

CHLORINE, LIQUID
Diamond Alkali Co., Pittsburgh, Pa.
Hooker Electrochemical Co., Buffalo Ave. and Ward St., Niagara Falls, N. Y.
Mathieson Alkali Wks., 60 E. 42nd St., New York City.
Niagara Alkali Co., 60 E. 42nd St., New York City.
Penn. Salt Mfg. Co., Widener Bldg., Philadelphia 7, Pa.
Pittsburgh Plate Glass Co., Columbia Chemical Division, Grant Bldg., Pittsburgh 19, Pa.
E. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.
Solvay Sales Corp., 40 Rector St., New York.

CHLORINE CONTROL EQUIPMENT
(See Residual Chlorine Control)

CHUTES, COAL, ETC.
Chicago Bridge & Iron Company, 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 300 W. Pershing Rd., Chicago 9, Ill.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

CLAMPS AND SLEEVES, PIPE
Amer. Cast Iron Pipe Co., Birmingham, Ala.
Carson-Cadillac Corp., Birmingham, Ala.
Dresser Mfg. Co., Bradford, Pa.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Iowa Valve Co., Hubbell Bldg., Des Moines, Iowa.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.
Warren Foundry & Pipe Co., 11 Broadway, New York City.

CLAMPS, BELL JOINT
Carson-Cadillac Corp., Birmingham, Ala.
Dresser Mfg. Div., Bradford, Pa.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Northrop & Co., Inc., Spring Valley, N. Y.

CLAMPS, PIPE REPAIR
Carson-Cadillac Corp., Birmingham, Ala.
Dresser Mfg. Div., Bradford, Pa.
Northrop & Co., Inc., Spring Valley, N. Y.

CLARIFIERS, SEWAGE AND WATER
American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dorr Co., 570 Lexington Ave., New York City.
Hardinge Co., Inc., York, Pa.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Lakeside Engrg. Corp., 222 W. Adams St., Chicago, Ill.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Walker Process Equip., Inc., Aurora, Ill.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

CLAY PIPE
(See Vitrified Clay Pipe and Products)

CLEANING COMPOUNDS
Oakite Products Corp., 73 Thames St., New York 6, N. Y.
C. B. Doyle Co., Westport, Conn.

CLEANING SEWAGE MAINS
(See Sewer Cleaning Equipment and Service)

CLEANING WATER MAINS
Flexible Underground Pipe Cleaning Co., 9059 Venice Blvd., Los Angeles, Calif.
National Water Main Cleaning Co., 30 Church St., New York City.
Pittsburgh Pipe Cleaner Co., 133 Dahlem St., Pittsburgh 6, Pa.

CLOROBEN
Cloroben Corp., Jersey City, N. J.

COAL & ASH HANDLING MACHINERY
(See Ash & Coal Handling Machy.)

COATINGS, ACID & ALKALI-PROOF
Atlas Mineral Products Co., Mertztown, Pa.
General Paint Corp., 3091 Mayfield Rd., Cleveland, Ohio.
Penn. Salt Mfg. Co., 1000 Widener Bldg., Philadelphia 7, Pa.
Philadelphia Quartz Co., Philadelphia, Pa.

COATINGS AND LININGS
(See Bituminous Coatings and Linings)

COCKS (Curb, Meter, Corporation, Etc.)
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Ford Meter Box Co., Wabash, Ind.
Hays Mfg. Co., Erie, Pa.
Mueller Co., Decatur, Ill.
Merco Nordstrom Valve Co., Lexington Ave., Pittsburgh, Pa.
Northrop & Co., Inc., Spring Valley, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

COLLECTORS (Sludge)
(See Sludge Collectors)

COMBUSTION INDICATORS AND RECORDERS
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Permutit Co., 330 W. 42nd St., York 18, N. Y.

COMMINUTORS
(See Grinders also)
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.

COMPARATORS, COLOR
Heilige, Inc., 3718 Northern Blvd., Long Island City, N. Y.
LaMotte Chem. Co., Towson 4, Md.
Photovolt Corp., 95 Madison Ave., New York 16, N. Y.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
W. A. Taylor & Co., 7300 York Road, Baltimore, Md.
Wallace & Tiernan Co., Inc., Newark, N. J.

COMPOUNDS, SEWER JOINT
Atlas Mineral Prod. Co., Mertztown, Pa.
Hydraulic Development Corp., 50 Church St., New York City.
Michael Hayman & Co., 870 E. Ferry St., Buffalo, N. Y.
Leadite Co., Girard Trust Co. Bldg., Philadelphia 2, Pa.
Servicised Products Corp., 6051 W. 65th St., Chicago, Ill.

COMPRESSORS, AIR
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Davey Compressor Co., Kent, Ohio

De Laval Steam Turbine Co., Trenton, N. J.
Jaeger Machine Co., 716 Dublin Ave., Columbus, Ohio.
Northrop & Co., Inc., Spring Valley, N. Y.
Roots-Connersville Blower Corp., Connersville, Ind.
Worthington Pump & Machinery Corp., Harrison, New Jersey.
Yeomans Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

CONCRETE BREAKERS
Northrop & Co., Inc., Spring Valley, N. Y.

CONCRETE CONTROL
Master Builders Corp., Cleveland, Ohio.

CONCRETE HARDENER
Diamond Alkali Co., Pittsburgh, Pa.
Philadelphia Quartz Co., Third & Ionic Sts., Philadelphia 6, Pa.

CONCRETE MIXERS
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.

CONCRETE PIPE
American Concrete Pipe Assn., 33 W. Grand Ave., Chicago 10, Ill.
Lock-Joint Pipe Co., P. O. Box 269, East Orange, N. J.

CONDENSERS (Steam)
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Amer. Cast Iron Pipe Co., Birmingham, Ala.
General American Process Equipment, 420 Lexington Ave., New York 17, N. Y.
U. S. Pipe & Foundry Co., Burlington, N. J.
Worthington Pump & Machy. Corp., Harrison, N. J.

CONTRACTORS (Water & Sewage Works)
Master Builders Co., Cleveland, Ohio.
Cement Gun Co., P. O. Box 424, Allentown, Pa.

CONTROLLERS, FLOW, LEVEL, PRESSURE, TEMPERATURE
Automatic Control Co., 1013 University Ave., St. Paul 4, Minn.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
B/W Controller Corp., Birmingham, Mich.
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Clark Controller Co., 1146 E. 152nd St., Cleveland, O.
Healy-Ruff Co., St. Paul, Minn.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
F. B. Leopold Co., Inc., 422 First Ave., Pittsburgh, Pa.
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland 13, Ore.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
R-S Products Co., Wayne Jct., Philadelphia 44, Pa.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

CONVEYORS AND CONVEYING EQUIPMENT
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dracco Corp., Cleveland, Ohio
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Mfg. Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Omega Machine Div., 9 Codding St., Providence, R. I.

COPPER, BRASS, BRONZE
American Brass Co., Waterbury, Conn.
Glauber Brass Mfg. Co., Kinsman, Ohio.
Hays Mfg. Co., Erie, Pa.
Mueller Brass Co., Port Huron, Mich.

COPPERAS (Sulfate of Iron)
Innis-Speiden & Co., 117 Liberty St., New York City.
E. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.

COPPER SULFATE
General Chemical Co., 40 Rector St., New York 6, N. Y.
Innis-Speiden & Co., 117 Liberty St., New York City.
Phelps Dodge Refg. Corp., 40 Wall St., New York, N. Y.
E. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.
Tennessee Corp., 621 Grant Bldg., Atlanta, Ga.

COPPER TUBE
American Brass Co., Waterbury, Conn.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Hays Mfg. Co., Erie, Pa.
Mueller Brass Co., Port Huron, Mich.
Mueller Co., Decatur, Ill.

CORROSION SUPPRESSION
Calgon, Inc., Hagan Bldg., Pittsburgh, Pa.
Electro Rust-Proofing Co., 29 W. Apple St., Dayton, Ohio.
Federal Telephone and Radio Co., Newark 1, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.

COUPLINGS & CONNECTIONS, PIPE
American Hard Rubber Co., 11 Mercer St., New York 13, N. Y.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
A. M. Byers Co., Pittsburgh, Pa.
Dresser Mfg. Div., Bradford, Pa.
Glauber Brass Mfg. Co., Kinsman, Ohio.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hays Mfg. Co., Erie, Pa.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Mueller Brass Co., Port Huron, Mich.
Mueller Co., Decatur, Ill.
National Tube Co., Frick Bldg., Pittsburgh, Pa.
Northrop & Co., Inc., Spring Valley, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

COUPLINGS, FLEXIBLE
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
De Laval Steam Turbine Co., Trenton, N. J.
Dresser Mfg. Div., Bradford, Pa.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 220 S. Belmont Ave., Indianapolis 6, Ind.
Peerless Pump Division, Food Machinery Corp., 301 West Avenue 26, Los Angeles 31, Calif.

COVERS, HANDHOLE, MANHOLE, SAMPLING
Josam Mfg. Co., 1783 East 11th St., Cleveland, Ohio.
Northrop & Co., Inc., Spring Valley, N. Y.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

CULVERT CLEANERS
W. H. Stewart, P. O. Box 767, Syracuse, N. Y.

CULVERT PIPE
(See Pipe, Culvert)

CURB BOXES
Central Foundry Co., 386 Fourth Ave., New York City.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Hays Mfg. Co., Erie, Pa.
Lee Hydraulic Co., Troy, N. Y.
Mueller Co., Decatur, Ill.
Northrup & Co., Inc., 50 Church St., New York 7, N. Y.

CURB STOPS
Glauber Brass Mfg. Co., Kinsman, Ohio.

DECHLORINATING EQUIPMENT AND CHEMICALS
American Water Softener Co., Philadelphia 33, Pa.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Everson Mfg. Co., 214 W. Huron St., Chicago, Ill.
General Chemical Co., 40 Rector St., New York, N. Y.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Omega Machine Co., Inc., 9 Codding St., Providence 1, R. I.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Virginia Smelting Co., West Norfolk, Va.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

DE-MINERALIZATION (Water)
Dorr Co., 570 Lexington Ave., New York 22, N. Y.
Infilco, Inc., 325 W. 25th Place, Chicago 16, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.

DIAPHRAGM, PUMP
(See Pumps, Diaphragm)

DIESEL ENGINES
(See Engines, Diesel)

DIFFUSERS—PLATES & TUBES
Carborundum Co., Perth Amboy, N. J.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.

DIGESTER EQUIPMENT (Sludge)
American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Dorr Co., 570 Lexington Ave., New York City.
Hardinge Co., York, Pa.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Walker Process Equip., Inc., Aurora, Ill.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

DISTRIBUTORS (Sewage Filter)
American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dorr Co., 570 Lexington Ave., New York City.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Yeomans Brothers Co., 1433 N Dayton St., Chicago 22, Ill.

DRAINS, GUTTER, FLOOR
James B. Clow Co., 201 N. Tallman St., Chicago, Ill.
Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio

DRILLING & BORING EQUIPMENT
Hydrauger Corp., Ltd., 116 New Montgomery St., San Francisco, Cal.

DRIVES, CHAIN
(See Chains)

DRIVES, RIGHT ANGLE GEAR
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Payne Dean Co., Madison, Conn.

DRY CHEMICAL FEEDERS
(See Chemical Feeding Equipment)

DRYERS, ROTARY
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
General American Process Equipment, 420 Lexington Ave., New York 17, N. Y.
Link-Belt Co., 300 W. Pershing Rd., Chicago 9, Ill

EJECTORS, PNEUMATIC OR STEAM (For Sewage and Sludge)
Yeomans Bros. Co., 1423 N. Dayton St., Chicago 22, Ill.

ELECTRICAL CONTROL EQUIPMENT
Clark Controller Co., 1146 E. 152nd St., Cleveland 10, Ohio.
General Electric Corp., Schenectady, N. Y.
Westinghouse Electric Corp., E. Pittsburgh, Pa.

ELECTRICALLY OPERATED GATE VALVES
(See Valves, Motor Operated)

ELEVATING AND CONVEYING EQUIPMENT
(See Conveyors & Conveying Equipment)

ENGINE-GENERATOR UNITS
Climax Engineering Co., Clinton, Iowa.
De Laval Steam Turbine Co., Trenton, N. J.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Homelite Corp., Port Chester, N. Y. (Gasoline Engine Driven and Portable.)
Northrop & Co., Inc., Spring Valley, N. Y. (Gasoline Driven Portable.)
Worthington Pump & Machy. Corp., Harrison, N. J.

ENGINES, DIESEL
Climax Engineering Co., Clinton, Iowa.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Worthington Pump & Machy. Corp., Harrison, N. J.

ENGINES, DUAL FUEL
Worthington Pump & Mach. Corp., Harrison, N. J.

ENGINES (Gas & Gasoline)
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Climax Engineering Co., Clinton, Iowa.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Worthington Pump & Machy. Corp., Harrison, N. J.

ENGINES, SLUDGE GAS
Climax Engineering Co., Clinton, Iowa.
Worthington Pump & Machy. Corp., Harrison, N. J.

EXPANSION JOINTS—PIPE
(See Joints—Expansion, Pipe)

EXHAUSTERS, AIR
(See Pump, Vacuum)

FANS, EXHAUST AND VENTILATING
Jeffrey Mfg. Co., The, 968-99 N. 4th St., Columbus 16, Ohio.

FEED, CHEMICAL
(See Chemical Feeding Equipment.)

FEED WATER FILTERS
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Water Softener Co., Philadelphia 33, Pa.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
F. B. Leopold Co., Inc., 422 First Ave., Pittsburgh, Pa.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
Worthington Pump & Machinery Corp., Harrison, N. J.

FEED WATER HEATERS
American Water Softener Co., Philadelphia 33, Pa.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Worthington Pump & Machy. Corp., Harrison, N. J.

FERRIC CHLORIDE
Diamond Alkali Co., Pittsburgh, Pa.
General Chemical Co., 40 Rector St., New York City.

Innis, Speiden & Co., 117 Liberty St., New York City.
Jordan Co., 51st St. & Merrimac Ave., Chicago, Ill.
Penn. Salt Mfg. Co., Widener Bldg., Philadelphia 7, Pa. (Liquid.)

FERRIC CHLORIDE FEEDERS
(See Chemical Feed and Proportioning Equipment)

FERRIC SULFATE
Tennessee Corp., 621 Grant Bldg., Atlanta, Ga.

FERROUS SULFATE
General Chemical Co., 40 Rector St., New York City.
Innis, Speiden & Co., 117 Liberty St., New York City.
R. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.

FIBRE PIPE
(See Pipe, Fibre)

FILTER ALUM
Activated Alum Corp., Curtis Bay, Baltimore, Md.
General Chemical Co., 40 Rector St., New York 6, N. Y.
Penn Salt Mfg. Co., Widener Bldg., Philadelphia 7, Pa.
R. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.

FILTERING EQUIPMENT
American Water Softener Co., Philadelphia 33, Pa.
American Well Works, Aurora, Ill.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
R. D. Cole Mfg. Co., Newnan, Ga.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Pacific Flush-Tank Co., 4241 Ravenswood Ave., Chicago 3, Ill.
General American Process Equipment, 420 Lexington Ave., New York 17, N. Y.
Hardinge Co., Inc., York, Pa.
Holyoke Mfg. Co., Holyoke, Mass.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
F. B. Leopold Co., Inc., 422 First Ave., Pittsburgh, Pa.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Roberts Filter Mfg. Co., Darby, Pa.
Sanequipment Engr. Co., 2843 Whitewood Drive, Ann Arbor, Mich.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.
Walker Process Equip. Co., Aurora, Ill.
Worthington Pump & Machinery Corp., Harrison, N. J.

FILTER PLATES AND TUBES
Carborundum Co., Perth Amboy, N. J.
Chicago Pump Co., 2336 Wolfram St., Chicago, Ill.

FILTER RATE CONTROLLERS & GAGES
(See Rate Controllers.)

FILTER UNDERDRAINS, SEWAGE
American Well Works, Aurora, Ill.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Infilco, Inc., 325 W. 25th Pl., Chicago 16, Ill.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.

FILTER UNDERDRAINS, WATER
American Water Softener Co., Philadelphia 33, Pa.
Carborundum Co., Niagara Falls, N. Y.
Cast Iron Pipe Research Assn., 122 S. Michigan Ave., Chicago, Ill.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
F. B. Leopold Co., Inc., 422 First Ave., Pittsburgh, Pa.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.

FILTER WASH CONTROL
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Hardinge Co., Inc., York, Pa.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Permutit Co., 330 W. 42nd St., York 18, N. Y.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.

FISH SCREEN
Electric Fish Screen Co., Hollywood 45, Calif.

FITTINGS, PIPE, SOLDER TYPE
American Brass Co., Waterbury, Conn.
Hays Mfg. Co., Erie, Pa.
Mueller Brass Co., Port Huron, Mich.
Northrop & Co., Inc., Spring Valley, N. Y.

FITTINGS, TEES, ELLS, ETC.
Albert Pipe Supply Co., Inc., Berry and N. 13th St., Brooklyn 11, N. Y.
American Cast Iron Pipe Co., Birmingham, Ala.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Carson-Cadillac Corp., Birmingham, Ala.
Central Foundry Co., 386 Fourth Ave., New York, N. Y.
Crane Co., 836 S. Michigan Av., Chicago, Ill.
Dresser Mfg. Div., Bradford, Pa.
Glauber Brass Mfg. Co., Kinsman, Ohio.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hays Mfg. Co., Erie, Pa.
James B. Clow Co., 201 N. Tallman Ave., Chicago, Ill.
M & H Valve & Fittings Co., anniston, Ala.
Mueller Brass Co., Port Huron, Mich.
Mueller Co., Decatur, Ill.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., Pittsburgh, Pa.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

FLASH MIXING EQUIPMENT
(See Mixers & Mixing Equipment)

FLEXIBLE JOINTS
(See Joints, Flexible Pipe.)

FLOATING COVERS (Digester)
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.

FLOCCULATING EQUIPMENT
American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., Milwaukee, Wis.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dorr Co., 570 Lexington Ave., New York City.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia, Pa.
Liquid Conditioning Corp., 114 Price St., Linden, N. J.
Omega Machine Co., Inc., 9 Codding St., Providence 1, R. I.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Stuart Corp., 516 N. Charles St., Baltimore, Md.
Vogt Mfg. Co., Louisville, Ky.
Walker Process Equip., Inc., Aurora, Ill.
Wallace & Tiernan Co., Inc., Newark 1, N. J.
Yeomans Bros. Co., 1423 N. Dayton St., Chicago 22, Ill.

FLOOR STANDS
American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Darling Valve Co., Williamsport, Pa.
Eddy Valve Co., Waterford, N. Y.
Infilco, Inc., 325 W. 25th Pl., Chicago 16, Ill.
Iowa Valve Co., Hubbell Bldg., Des Moines, Iowa.
F. B. Leopold Co., 422 First Ave., Pittsburgh, Pa.
Ludlow Valve Mfg. Co., Troy, N. Y.
M & H Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Merco Nordstrom Valve Co., Lexington Ave., Pittsburgh, Pa.
Rensselaer Valve Co., Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

FLOW METERS
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Northrop & Co., Inc., Spring Valley, N. Y.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roots-Connersville Blower Corp., Connersville, Ind.
Simplex Valve & Meter Co., Philadelphia, Pa.
R. W. Sparling, Box 3277 Terminal Annex, Los Angeles, Calif.

FLOW RECORDERS
(See Rate of Flow Recorders)

FLOW REGULATORS
(See Rate of Flow Controllers)

FLUMES, IRON & STEEL
Chicago Bridge & Iron Company, 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

FOOT VALVE WITH STRAINER
(See Valves, Foot)

FORMS & GASKETS FOR PIPE JOINTS
Johns-Manville, 22 E. 40th St., New York City.
L. A. Weston, Adams, Mass.

FURNACES, MELTING, POURING LADLES, POTS, ETC.
Atlas Mineral Products Co., Mertztown, Pa.
Hydraulic Development Corp., 50 Church St., New York City.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.

GAGES (Liquid Level, Pressure Recording)
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Everson Mfg. Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland 13, Ore.
Northrop & Co., Inc., Spring Valley, N. Y.
Pacific Flush-Tank Co., 4241 Ravenswood Ave., Chicago 3, Ill.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.
Wallace & Tiernan Co., Inc., Newark, N. J.

GARBAGE GRINDERS
(See Grinders)

GAS DIFFUSERS
American Well Works, Aurora, Ill.
Carborundum Co., Perth Amboy, N. J.
Chicago Pump Co., 2336 Wolfram St., Chicago, Ill.
Dorr Co., 570 Lexington Ave., New York City.
Vogt Mfg. Co., Louisville, Ky.
Walker Process Equip., Inc., Aurora, Ill.

GAS ENGINES
(See Engines, Gas & Gasoline)

GAS HOLDERS
Chicago Bridge & Iron Company, 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
Dresser Industries, Inc., Cleveland 13, Ohio.
Graver Tank & Mfg. Co., Inc., East Chicago, Ind.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

GAS INDICATORS AND DETECTORS
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.

GATES, SHEAR
Chapman Valve Mfg. Co., 203 Hampshire St., Indian Orchard, Mass.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Iowa.
Ludlow Valve Mfg. Co., Troy, N. Y.
M & H Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Rensselaer Valve Co., Troy, N. Y.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

GATES, SLUICE
Filer and Stowell Co., Milwaukee 7, Wis.
Chapman Valve Mfg. Co., 203 Hampshire St., Indian Orchard, Mass.
Iowa Valve Co., Hubbell Bldg., Des Moines, Iowa.
Ludlow Valve Mfg. Co., Troy, N. Y.
Mueller Co., Chattanooga, Tenn.
Rensselaer Valve Co., Troy, N. Y.
S. Morgan Smith Co., York, Pa.

GATE VALVE OPERATOR
Lee Hydraulic Co., Troy, N. Y.
Payne Dean & Co., Madison, Conn.

GATE VALVES
(See Valves, Gate)

GEARS, SPEED REDUCING
De Laval Steam Turbine Co., Trenton, N. J.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia, Pa.
Worthington Pump & Machy. Corp., Harrison, N. J.

GENERATORS
General Electric Co., Schenectady, N. Y.
Westinghouse Electric Corp., Pittsburgh, Pa.

GENERATOR SETS, GAS & DIESEL ENGINES
(See Engine Generating Units)

GOOSENECKS
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hays Mfg. Co., Erie, Pa.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

GREASE INTERCEPTORS
Josam Mfg. Co., 1783 E. 11th St., Cleveland, O.

GREEN SAND (Zeolite)
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., York 18, N. Y.

Roberts Filter Mfg. Co., Darby, Pa.
Zeolite Chemical Co., 140 Cedar New York 6, N. Y.

GRINDERS, GARBAGE & SCREENINGS
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dorr Co., 570 Lexington Ave., New York City.
General Electric Co. (household), Bridgeport, Conn.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 Hunting Park Ave., Philadelphia, Pa.
Westinghouse Elect. Corp. (household), Pittsburgh, Pa.

GRIT CHAMBER EQUIPMENT
American Well Works, Aurora, Ill.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dorr Co., 570 Lexington Ave., New York City.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Walker Process Equip., Inc., Aurora, Ill.

GROUND KEY STOPS
Glauber Brass Mfg. Co., Kinsman, Ohio.

GUNITING CONTRACTING
Cement Gun Co., P. O. Box 424, Allentown, Pa.

HOSE, STEAM, AIR, WATER, SUCTION
American Brass Co., Waterbury, Conn.
Edson Corp., 49 D St., So. Boston, Mass.
Jaeger Machine Co., 716 Dublin Ave., Columbus, Ohio.
Northrop & Co., Inc., Spring Valley, N. Y.

HYDRANT PUMPS
(See Pumps, Hydrant)

HYDRANTS (Fire)
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Darling Valve Co., Williamsport, Pa.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
James B. Clow Co., 201 N. Tallman St., Chicago, Ill.
Ludlow Valve Mfg. Co., Troy, N. Y
M & H Valve & Fittings Co., Anniston, Ala.
Mueller Co., Decatur, Ill.
Rensselaer Valve Co., Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

HYDRAULICALLY OPERATED GATE VALVES
(See Valves, Gate)

HYDROGEN ION EQUIPMENT
(See pH Test Equipment.)

HYPO-CHLORINATORS
American Water Softener Co., Philadelphia 33, Pa.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Milton Roy Co., Chestnut Hill, Philadelphia 18, Pa.
Omega Mach. Co., Inc., 9 Codding St., Providence 1, R. I.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Phipps & Bird, Inc., 915 E. Cary St., Richmond, Va.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Wallace & Tiernan Co., Newark 1, N. J.

HYPOCHLORITES (High Test)
(See Calcium Hypochlorite)

INCINERATORS
Dorr Co., Inc., 570 Lexington Ave., New York City.
Nichols Engr. & Research Corp., 60 Wall Tower, New York City.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

INDICATORS, LARGE DIAL, ILLUMINATED
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, O.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Healy-Ruff Co., St. Paul, Minn.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland 13, Ore.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
R. W. Sparling, Box 3277 Terminal Annex, Los Angeles, Calif.

INSTRUMENTS (Meteorological)
M. C. Stewart, Ashburnham, Mass.

INSTRUMENTS (Surveying, Hydrographic and Navigation)
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland 13, Oregon.

INSULATION
Carborundum Co., Perth Amboy, N. J.
Ford Meter Box Co., Wabash, Ind. (Water Meter).
Johns-Manville, 22 E. 40th St., New York City.

IRON SULFATE
(See Ferric Sulfate, or Ferrous Sulfate)

JOINTING COMPOUND AND MATERIALS
Atlas Mineral Prod. Co., Mertztown, Pa.
J. H. Grancell, 1601 E. Nadeau St., Los Angeles 1, Calif.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hydraulic Development Corp., 50 Church St., New York City.
Inertol Co., Newark, N. J.
Leadite Co., Philadelphia, Pa.
Michael Hayman & Co., 870 E. Ferry St., Buffalo, N. Y.
Northrop & Co., Inc., Spring Valley, N. Y.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Penn. Salt Mfg. Co., 1000 Widener Bldg., Philadelphia 7, Pa.
Rubber and Plastics Compound Co., 30 Rockefeller Plaza, New York, N. Y.
Servicised Products Corp., 6051 West 65th St., Chicago, Ill.
Wailes Dove-Hermiston, Westfield, N. J.

JOINTS, EXPANSION—PIPE
Amer. Cast Iron Pipe Co., Birmingham, Ala.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Central Foundry Co., 386 Fourth Ave., N. Y., N. Y.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Dresser Mfg. Div., Bradford, Pa.
Josam Mfg. Co., 1783 East 11th St., Cleveland, Ohio.
Lock Joint Pipe Co., P. O. Box 269, East Orange, N. J.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Co., 11 Broadway, New York City.

JOINTS, MECHANICAL, FLEXIBLE
American Cast Iron Pipe Co., Birmingham, Ala.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Carson-Cadillac Corp., Birmingham, Ala.
Central Foundry Co., 386 Fourth Ave., New York City.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dresser Mfg. Div., Bradford, Pa.
Hays Mfg. Co., Erie, Pa.
Lock Joint Pipe Co., P. O. Box 269, East Orange, N. J.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.
R. D. Wood Co., 400 Chestnut St., Philadelphia, Pa.

JUTE, BRAIDED AND TWISTED
Atlas Mineral Prod. Co., Mertztown, Pa.
Hydraulic Development Corp., 50 Church St., New York City.
Johns-Manville, 22 E. 40th St., New York City.
Northrop & Co., Inc., Spring Valley, N. Y.

LABORATORY APPARATUS AND REAGENTS
Chemlab Specialties Co., 52 Alvarado Road, Berkeley, Calif.
Hellige, Inc., 3718 Northern Blvd., Long Island City, N. Y.
Klett Mfg. Co., 179 E. 87th St., New York 28, N. Y.
LaMotte Chem. Co., Towson 4, Md.
Nat'l Technical Labs, Los Angeles, Calif.
Permutit Co., 330 W. 42nd St., New York City.
Phipps & Bird, Inc., Sixth & Byrd, Richmond, Va.
W. A. Taylor & Co., 7300 York Rd., Baltimore, Md.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

LEAK LOCATING INSTRUMENTS
(See Water Leak Locating Instruments)

LIGHTING
General Electric Co., Schenectady, N. Y.
Westinghouse Electric Corp., Pittsburgh, Pa.

LIMES
National Gypsum Co., Buffalo, N. Y.

LIQUID CHLORINE
(See Chlorine, Liquid.)

LOCATORS, BOX, PIPE AND VALVE
Globe Phone Co., Reading, Mass.
Fischer Research Co., Los Angeles, Cal.

MAN HOLE FRAMES, COVERS
James B. Clow Co., 201 N. Tallman St., Chicago, Ill.

MANOMETERS
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Lakeside Engineering Corp., 222 W. Adams St., Chicago, Ill.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

METER ACCESSORIES, BOXES, HOUSING SETTING, ETC.
Badger Meter Co., Milwaukee, Wis.
Buffalo Meter Co., 2909 Main St., Buffalo, N. Y.
Ford Meter Box Co., Wabash, Ind.
Glauber Brass Mfg. Co., Kinsman, Ohio.
Meter Specialty Co., 1332 N. 7th St., Memphis, Tenn.
Mueller Co., Decatur, Ill.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Illinois.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.

METER COUPLINGS & YOKES
Badger Meter Mfg. Co., Milwaukee, Wis.
Buffalo Meter Co., 2909 Main St., Buffalo, N. Y.
Ford Meter Box Co., Wabash, Ind.
Glauber Brass Mfg. Co., Kinsman, Ohio.
Hays Mfg. Co., Erie, Pa.
Mueller Co., Decatur, Ill.
Neptune Meter Co., 50 West 50th St., New York City.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.

METERS, ELECTRIC
General Electric Co., Schenectady, N. Y.
Westinghouse Electric Corp., Pittsburgh, Pa.

METERS, GAS
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, O.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
Roots-Connersville Blower Corp., Connersville, Ind.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.

METER TESTING EQUIPMENT
Badger Meter Mfg. Co., Milwaukee, Wis.
Buffalo Meter Co., 2909 Main St., Buffalo, N. Y.
Ford Meter Box Co., Wabash, Ind.
Neptune Meter Co., 50 W. 50th St., New York City.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
Worthington - Gamon Meter Co., Newark, N. J.

METERS, VENTURI
Bailey Meter Co., 1072 Invanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Simplex Valve & Meter Co., 68th & Upham Sts., Philadelphia, Pa.

METERS, WATER & SEWAGE
Badger Meter Mfg. Co., Milwaukee, Wis.
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Buffalo Meter Co., 2909 Main St., Buffalo, N. Y.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Everson Mfg. Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Neptune Meter Co., 50 W. 50th St., New York City.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
R. W. Sparling, Box 3277 Terminal Annex, Los Angeles, Calif.
Worthington - Gamon Meter Co., Newark, N. J.

METERS (Water), CLEANING SOLUTION
Skasol Corp., Webster Groves, Mo.

MIXERS AND MIXING EQUIPMENT
(Chemical, Water, and Sewage)
American Well Works, Aurora, Ill.
Carborundum Co., Perth Amboy, N. J.
Ralph B. Carter Co., 53 Park Place, New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dorr Co., 570 Lexington Ave., New York City.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
General American Process Equipment, 420 Lexington Ave., New York 17, N. Y.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Lakeside Engrg. Corp., 222 W. Adams St., Chicago, Ill.
F. B. Leopold Co., 422 First Ave., Pittsburgh, Pa.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Omega Mach. Co., Inc., 9 Codding St., Providence 1, R. I.
Permutit Co., 330 W. 42nd St. New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Vogt Mfg. Co., Louisville, Ky.
Walker Process Equip. Co., Aurora, Ill.
Wallace & Tiernan Co., Inc., Newark, N. J.

MOTORS, ELECTRIC
Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.

Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
General Electric Co., Schenectady, N. Y.
Westinghouse Electric Corp., P. O. Box 868, Pittsburgh 30, Pa.

NIPPLES, PIPE

American Hard Rubber Co., 11 Mercer St., New York 13, N. Y.
A. M. Byers Co., Pittsburgh, Pa.
Dresser Mfg. Co. Div., Bradford, Pa.
Glauber Brass Mfg. Co., Kinsman, Ohio.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hays Mfg. Co., Erie, Pa.
Mueller Brass Co., Port Huron, Mich.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.

NOZZLES, FLOW

Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.

NOZZLES, SPRAY

Chain Belt Co., Milwaukee, Wis.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Mueller Brass Co., Port Huron, Mich.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Yeomans Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

OPERATING TABLES (Filter)

Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
F. B. Leopold Co., 422 First Ave., Pittsburgh, Pa.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.

ORIFICES

Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Infilco, Inc., 325 W. 25th Pl., Chicago 16, Ill.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.

PACKING

Johns-Manville, 22 E. 40th St., New York City.
Northrop & Co., Inc., Spring Valley, N. Y.

PAINT, ACID RESISTING

Atlas Mineral Prod. Co,, Mertztown, Pa.
General Paint Corp., 3091 Mayfield Rd., Cleveland, Ohio
Inertol Co., Newark 5, N. J.
Penn. Salt Mfg. Co., 1000 Widener Bldg., Philadelphia 7, Pa.
Wailes Dove-Hermiston, Westfield, N. J.

PENSTOCKS

Chicago Bridge & Iron Co., 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

pH TEST EQUIPMENT

American Water Softening Co., Philadelphia 33, Pa.
Hellige, Inc., 3718 Northern Blvd., Long Island City, N. Y.
Beckman Instruments Div., National Technical Laboratories, So. Pasadena, Calif.
LaMotte Chem. Co., Towson 4, Balto, Md.
Phipps & Bird, Inc., 303 S. 6th St., Richmond 5, Va.
Photovolt Corp., 95 Madison Ave., New York 16, N. Y.
W. A. Taylor & Co., 7300 York Rd., Baltimore, Md.
Wallace & Tiernan Co., Inc., Newark, N. J.

PIPE AND TANK COATINGS

General Paint Corp., 3091 Mayfield Rd., Cleveland, Ohio.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.

PIPE, ASBESTOS-CEMENT

Johns-Manville Co., 22 E. 40th St., New York City.

PIPE BENDING MACHINES

Hays Mfg. Co., Erie, Pa.

PIPE, CAST IRON

Albert Pipe Supply Co., Berry and N. 13th St., Brooklyn 11, N. Y.
American Cast Iron Pipe Co., Birmingham, Ala.
Central Foundry Co., 386 4th Ave., New York City.
James B. Clow Co., 201 N. Tallman St., Chicago, Ill.
Hunt, Rodney, Machine Co., Orange, Mass.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

PIPE, CEMENT LINED

American Cast Iron Pipe Co., Birmingham, Ala.
Central Foundry Co., 386 Fourth Ave., New York City.
Centrilline Corp., 140 Cedar St., New York City.
Preload Enterprises, Inc., 420 Lexington Ave., New York, N. Y.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.

PIPE, CEMENT LINING OF

Centrilline Corp., 140 Cedar St., New York City.
Lock Joint Pipe Co., P. O. Box 269, East Orange, N. J.
Preload Enterprises, Inc., 420 Lexington Ave., New York, N. Y.

PIPE, CONCRETE, PRESSURE

Lock Joint Pipe Co., P. O. Box 269, East Orange, N. J.

PIPE, COPPER, BRASS AND BRONZE

American Brass Co., Waterbury, Conn.
Hays Mfg. Co., Erie, Pa.
Mueller Brass Co., Port Huron, Mich.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.

PIPE, CULVERT

Cast Iron Pipe Research Assn., 122 S. Michigan Ave., Chicago.
Central Foundry Co., 386 Fourth Ave., New York City.
Johns-Manville, 22 E. 40th St., New York City.
Lock Joint Pipe Co., P. O. Box 269, East Orange, N. J.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

PIPE CUTTING MACHINES

Northrop & Co., Inc., Spring Valley, N. Y.
Ridge Tool Co., Elyria, Ohio.
The A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

PIPE, DRILLS, JACKS AND PUSHERS

Northrop & Co., Inc., Spring Valley, N. Y.

PIPE ENAMELS, PROTECTIVE —COAL TAR

General Paint Corp., 3091 Mayfield Rd., Cleveland, Ohio.
Inertol Co., Newark 5, N. J.
Wailes Dove-Hermiston, Westfield, N. J.

PIPE, FELT WRAPPING

General Paint Corp., 3091 Mayfield Rd., Cleveland, Ohio.
Johns-Manville, 22 E. 40th St., New York City.

PIPE FITTINGS (See Fittings

PIPE, HARD RUBBER LINED

American Hard Rubber Co., 11 Mercer St., New York 13, N. Y.

PIPE JOINT COMPOUNDS

(See Jointing Compounds.)

PIPE JOINTS (Mechanical)

Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Carson-Cadillac Corp., Birmingham, Ala.
Central Foundry Co., 386 Fourth Ave., New York, N. Y.
Dresser Mfg. Div., Bradford, Pa.
Hays Mfg. Co., Erie, Pa.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
Warren Foundry & Pipe Co., 11 Broadway, New York City.
L. A. Weston, Adams, Mass.
R. D. Wood & Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.
U. S. Pipe & Foundry Co., Burlington, N. J.

PIPE JOINT TOOLS

Atlas Mineral Products Co., Mertztown, Pa.
Hays Mfg. Co., Erie, Pa.
Hydraulic Development Corp., 50 Church St., New York City.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.

PIPE, LEAD LINED

Nooter, John, Boiler Works Co., 1410 S. 2nd St., St. Louis 4, Mo.

PIPE LOCATORS

Fisher Research Lab., 1961 University Ave., Palo Alto, Calif.
Globe Phone Mfg. Co., Reading, Mass.
Goldak Co., 1544 W. Glenoaks Blvd., Glendale 1, Calif.
Northrop & Co., Inc., Spring Valley, N. Y.

PIPE PUSHERS

Northrop & Co., Inc., Spring Valley, N. Y.
Hydrauger Corp., Ltd., 116 New Montgomery St., San Francisco, Calif.

PIPE, SEWER

American Cast Iron Pipe Co., Birmingham, Ala.
Am. Concrete Pipe Assn., 33 W. Grand Ave., Chicago, Ill. (Reinforced Concrete.)
Cast Iron Pipe Research Assn., 122 S. Michigan Ave., Chicago, Ill.
Central Foundry Co., 386 Fourth Ave., New York City.
Johns-Manville, 22 E. 40th St., New York City.
Lock Joint Pipe Co., P. O. Box 269, East Orange, N. J.
Nat'l Clay Pipe Mfrs., 111 W. Washington St., Chicago, Ill.
U. S. Pipe & Foundry Co., Burlington, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.

PIPE, STEEL (All Types)

American Rolling Mill Co., Middletown, Ohio.
Chicago Bridge & Iron Company, 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Nooter, John, Boiler Works Co., 1410 S. 2nd St., St. Louis 4, Mo.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

PIPE TAPPING MACHINES

Hays Mfg. Co., Erie, Pa.
Mueller Co., Decatur, Ill.
The A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

PIPE TEST PLUGS

Northrop & Co., Inc., Spring Valley, N. Y.
Richmond Fdry. & Mfg. Co., Richmond, Va.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

PIPE TOOLS (Cutters, Extractors, Threaders, Wrenches, Etc.)

Ridge Tool Co., Elyria, Ohio.

PIPE, VITRIFIED CLAY

(See Vitrified Clay Pipe and Products.)

PIPE, WRAPPING

General Paint Corp., 3091 Mayfield Rd., Cleveland, Ohio.
Johns-Manville, 22 E. 40th St., New York City.

PIPE, WROUGHT IRON

A. M. Byers Co., Clark Bldg., Pittsburgh, Pa.

PITOT TUBES

Pitometer Company, 50 Church St., New York City.
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland, Ore.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.

PNEUMATIC CONVEYORS FOR WATER AND SEWAGE TREATMENT

Dracco Corp., Cleveland, Ohio.

POROUS TUBES AND PLATES

Carborundum Co., Niagara Falls, N. Y.
Chicago Pump Co., 2336 Wolfram St., Chicago, Ill.

PRESSURE REGULATORS

(See Regulators, Pressure)

PROPORTIONING EQUIPMENT

(See Chemical Feed and Proportioning Equip.)

PUMPING CONTROLS

Automatic Control Co., 1031 University Ave., St. Paul 4, Minn.
Healy-Ruff Co., St. Paul, Minn.
B/W Controller Corp., Birmingham, Mich.

PUMP PRIMERS

Barrett, Haentjens & Co., Hazleton, Pa.
Ralph B. Carter Co., 53 Park Place, New York City.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Roots-Connersville Blower Corp., Connersville, Ind.
Valve and Primer Corp., 30 W. Washington St., Chicago 2, Ill.
Yeoman Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, ACID-HANDLING

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Hard Rubber Co., 11 Mercer St., New York 13, N. Y.
Aurora Pump Co., Aurora, Ill.
Ralph B. Carter Co., 53 Park Place, New York City.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dayton-Dowd Pumps, Peerless Pump Division, Inc., Quincy, Illinois.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Milton Roy Co., Chestnut Hill, Philadelphia, Pa.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.
Wallace & Tiernan Co., Inc., Newark, N. J.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeoman Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, CELLAR DRAINER

(See Pumps, Sump)

PUMPS, CENTRIFUGAL

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Well Works, Aurora, Ill.
Aurora Pump Co., Aurora, Ill.
Barrett, Haentjens & Co., Hazleton, Pa.
Ralph B. Carter Co., 53 Park Place, New York City.
Chain Belt Co., Milwaukee, Wis.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dayton - Dowd Pumps, Peerless Pump Division, Inc., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Domestic Engine & Pump Co., Shippensburg, Pa.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Gorman-Rupp Co., Mansfield, Ohio.
Homelite Corp., Port Chester, N. Y.
Layne & Bowler, Inc., P. O. Box 215, Hollywood Sta., Memphis 8, Tenn.
Morris Machine Works, Baldwinsville, N. Y.
Northrop & Co., Inc., Spring Valley, N. Y.

Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 Dayton St., Chicago 22, Ill.

PUMPS, DEEP WELL

American Well Works, Aurora, Ill.
Aurora Pump Co., Aurora, Ill.
A. D. Cook Co., Inc., Lawrenceburg, Indiana.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Layne & Bowler, Inc., P. O. Box 215, Hollywood Sta., Memphis 8, Tenn.
Morris Machine Works, Baldwinsville, N. Y.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Worthington Pump & Mach. Corp., Harrison, N. J.

PUMPS, DIAPHRAGM

Ralph B. Carter Co., 53 Park Place, New York City.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Dorr Co., 570 Lexington Ave., New York City.
Edson Corp., 49 D. St., South Boston, Mass.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Marlow Pumps, Inc., 412 Greenwood Ave., Ridgewood, N. J.
Milton Roy Co., Chestnut Hill, Philadelphia 18, Pa.
Northrop & Co., Inc., Spring Valley, N. Y.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Wallace & Tiernan Co., Inc., Newark, N. J.
Yeomans Brothers Co., 1433 Dayton St., Chicago 22, Ill.

PUMPS, DRAINAGE (See Pumps, Sewage and Drainage)

PUMPS, HYDRANT

Edson Corp., 49 D St., South Boston, Mass.
Northrop & Co., Inc., Spring Valley, N. Y.
Yeomans Brothers Co., 1433 Dayton St., Chicago 22, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.

PUMPS, HYDRAULIC

Ross Valve Mfg. Co. Inc., Troy, N. Y.

PUMPS, JETTING

Ralph B. Carter Co., 53 Park Pl., New York N. Y.
Domestic Engine & Pump Co., Shippensburg, Pa.
Gorman-Rupp Co., Mansfield, Ohio.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Div., Food Machinery Corp., 301 W. Ave. 26, Los Angeles 31, Calif.

PUMPS, PORTABLE

Barrett, Haentjens & Co., Hazleton, Pa.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dayton - Dowd Pumps, Peerless Pump Div., Inc., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Domestic Engine & Pump Co., Shippensburg, Pa.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Gorman-Rupp Co., Mansfield, Ohio.
Homelite Corp., Port Chester, N. Y.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Division Food Machinery Corp., 301 West Avenue 26, Los Angeles, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, ROTARY

De Laval Steam Turbine Co., Trenton, N. J.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Layne & Bowler, Inc., P. O. Box 215, Hollywood Station, Memphis 8, Tenn.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Roots-Connersville Blower Corp., Connersville, Ind.
Worthington Pump & Mach. Corp., Harrison, N. J.

PUMPS, SELF PRIMING

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Aurora Pump Co., Aurora, Ill.
Barrett, Haentjens & Co., Hazleton, Pa.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dayton - Dowd Pumps, Peerless Pump Div., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Domestic Engine & Pump Co., Shippensburg, Pa.
Gorman-Rupp Co., Mansfield, Ohio.
Homelite Corp., Port Chester, N. Y.
Layne & Bowler, Inc., P. O. Box 215, Hollywood Sta., Memphis 8, Tenn.
Marlow Pumps, 412 Greenwood Ave., Ridgewood, N. J.
Peerless Pump Div., Food Machinery Corp., 301 W. Ave. 26, Los Angeles 31, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Roots-Connersville Blower Corp., Connersville, Ind.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, SEWAGE AND DRAINAGE

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Well Works, Aurora, Ill.
Aurora Pump Co., Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dayton - Dowd Pumps, Peerless Pump Div., Inc., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Domestic Engine & Pump Co., Shippensburg, Pa.
Edson Corp., 49 D St., South Boston, Mass.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago, Ill.
Gorman-Rupp Co., Mansfield, Ohio.
Marlow Pumps, 412 Greenwood Ave., Ridgewood, N. J.
Morris Machine Works, Baldwinsville, N. Y.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue 26, Los Ageles 31, Calif.
Pomona Pump Co., Pomona, Calif.
S. Morgan Smith Co., York, Pa.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, SLUDGE

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Well Works, Aurora, Ill.
Aurora Pump Co., Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dayton - Dowd Pumps, Peerless Pump Div., Inc., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Dorr Co., 570 Lexington Ave., New York City.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Infilco, Inc., 325 W. 25th Pl., Chicago, Ill.
Marlow Pumps, 412 Greenwood Ave., Ridgewood, N. J.
Morris Machine Works, Baldwinsville, N. Y.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Peerless Pump Division Food Machinery Corp., 301 West Avenue 26, Los Angeles, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, SUMP (AND CELLAR DRAINER)

American Well Works, Aurora, Ill.
Aurora Pump Co., Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dayton - Dowd Pumps, Peerless Pump Div., Inc., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Domestic Engine & Pump Co., Shippensburg, Pa.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Gorman-Rupp Co., Mansfield, Ohio.
Morris Machine Works, Baldwinsville, N. Y.
Peerless Pump Division Food Machinery Corp., 301 West Avenue 26, Los Angeles, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, TURBINE

American Well Works, Aurora, Ill.
Aurora Pump Co., Aurora, Ill.
Dayton - Dowd Pumps, Peerless Pump Div., Inc., Quincy, Ill.
De Laval Steam Turbine Co., Trenton, N. J.
Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Layne & Bowler, Inc., P. O. Box 215, Hollywood Sta., Memphis 8, Tenn.
Morris Machine Works, Baldwinsville, N. Y.
Peerless Pump Division Food Machinery Corp., 301 West Avenue 26, Los Angeles 31, Calif.
Pomona Pump Co., 206 E. Commercial St., Pomona, Calif.
S. Morgan Smith Co., York, Pa.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

PUMPS, VACUUM

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Barrett, Haentjens & Co., Hazleton, Pa.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Roots-Connersville Blower Corp., Connersville, Ind.
Vogt Brothers Mfg. Co., Louisville, Ky.
Worthington Pump & Mach. Corp., Harrison, N. J.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

RAIN GAGES

M. C. Stewart, Ashburnham, Mass.

RATE OF FLOW CONTROLLERS

Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Everson Mfg. Co., 214 W. Huron St., Chicago, Ill.
Healy-Ruff Co., St. Paul, Minn.
Infilco, Inc., 325 W. 25th Place., Chicago, Ill.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
S. Morgan Smith Co., York, Pa.

RATE OF FLOW RECORDERS

Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland 13, Ore.
Northrop & Co., Inc., Spring Valley, N. Y.
Permutit Co., 330 W. 42nd St., New York City.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
R. W. Sparling, Box 3277 Terminal Annex, Los Angeles, Calif.

REAMERS

Mueller Co., Decatur, Ill.
Ridge Tool Co., Elyria, Ohio.

RECARBONATORS

(See Carbonators)

REFRACTORIES

Carborundum Co., Perth Amboy, N. J.
Johns-Manville, 22 E. 40th St., New York City.

REGULATORS, PRESSURE

(Water and Gas)

American Radiator Co., Bessemer Bldg., Pittsburgh, Pa.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Clark Controller Co., 1146 E. 152nd St., Cleveland 10, Ohio.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Healy-Ruff Co., St. Paul, Minn.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Peerless Pump Division, Food Machinery Corp., 301 W. Avenue, Los Angeles 26, Calif.
Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
S. Morgan Smith Co., York, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

RESIDUAL CHLORINE CONTROL APPARATUS

Hellige, Inc., 3718 Northern Blvd., Long Island City, N. Y.
LaMotte Chem. Co., Towson 4, Md.
W. A. Taylor & Co., 7300 York Rd., Baltimore, Md.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

RUBBER LINED PIPE AND VALVES

American Hard Rubber Co., 11 Mercer St., New York 13, N. Y.

RUST PREVENTION

(Tanks, Pipe Lines, Submerged Metal)

Calgon, Inc., Hagan Bldg., Pittsburgh, Pa.
Electro Rust-Proofing Corp., 1026 Wayne Ave., Dayton 10, Ohio. (Cathodic Protection)
Inertol Co., Newark 5, N. J.
Johnston & Jennings Co., Rusta Restor Div., 877 Addison Rd., Cleveland, Ohio.
Philadelphia Quartz Co., Third & Ionic Sts., Philadelphia, Pa.
Wailes Dove-Hermiston, Westfield, N. J.

SADDLES, PIPE

Clow & Sons, James B., 201 N. Talman Ave., Chicago, Ill.
Dresser Mfg. Div., Bradford, Pa.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Hays Mfg. Co., Erie, Pa.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.

SAMPLERS AND SAMPLING TABLES

Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Drake Mfg. Co., 1461 Belfield Ave., Philadelphia, Pa.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
F. B. Leopold Co., 422 First Ave., Pittsburgh, Pa.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Roberts Filter Mfg. Co., Darby, Pa.

SAND EXPANSION INDICATORS

Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.

SCALES, SLUDGE CONVEYOR

Builders-Providence, 9 Codding St., Providence, R. I.

SCALES, WEIGHING

Fairbanks-Morse Co., 600 S. Michigan Ave., Chicago 5, Ill.

SCREENINGS GRINDERS

(See Grinders)

SCREENS, INTAKE

Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.

Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 300 W. Pershing Rd., Chicago 9, Ill.
Yeomans Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

SCREENS, SAND, GRAVEL AND STONE

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 300 W. Pershing Rd., Chicago, Ill.

SCREENS, SEWAGE

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Dorr Co., 570 Lexington Ave., New York City.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Walker Process Equip. Co., Aurora, Ill.
Yeomans Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

SCREENS, VIBRATING

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Link Belt Co., 307 N. Michigan Ave., Chicago 1, Illinois.

SCREENS, WATER WELL

Layne & Bowler, Inc., P. O. Box 215, Hollywood Sta., Memphis 8, Tenn.

SEGMENT BLOCKS AND PIPE LINER PLATES (Vitrified)

Nat'l Clay Pipe Mfrs. Inc., 111 W. Washington St., Chicago, Ill.

SEPTIC TANKS (Vitrified Clay)

Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.

SERVICE BOXES

Central Foundry Co., 386 Fourth Ave., New York City.
Hays Mfg. Co., Erie, Pa.
Lee Hydraulic Co., Troy, N. Y.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.

SERVICE PIPE

Glauber Brass Mfg. Co., Kinsman, Ohio.

SEWAGE EJECTORS

(See Ejectors, Pneumatic.)

SEWAGE SAMPLERS

Chicago Pump Co., 2349 Wolfram St., Chicago, Ill.
Drake Mfg. Co., 1461 Belfield Ave., Philadelphia, Pa.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Omega Machine Co., 9 Codding St., Providence 1, R. I.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Wallace & Tiernan Co., Inc., Newark, N. J.

SEWAGE SLUDGE INCINERATOR

Dorr Co., 570 Lexington Ave., New York City.
Nichols Engineering & Research Corp., 60 Wall Tower, New York City.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

SEWER CLEANING EQUIP. AND SERVICE

J. C. Fitzgerald, P. O. Box 280, Coral Gables 34, Fla.
Flexible Sewer-Rod Eqpt. Co., 9059 Venice Blvd., Los Angeles, Calif.
National Water Main Cleaning Co., 30 Church St., New York City.
Pittsburgh Pipe Cleaner Co., 133 Dahlem St., Pittsburgh 6, Pa.
W. H. Stewart, P. O. Box 767, Syracuse, N. Y.
Turbine Sewer Machine Co., 5210 W. State St., Milwaukee, Wis.

SEWER ODOR PREVENTION

(See Chlorine)
Cloroben Corp., Jersey City, N. J.

SEWER PIPE

(See Pipe, Sewer)

SEWER PIPE JOINTS

Atlas Mineral Prod. Co., Mertztown, Pa.
Michael Hayman & Co., 870 E. Ferry St., Buffalo, N. Y.
Hydraulic Development Corp., 50 Church St., New York City.
Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.
L. A. Weston, Adams, Mass.

SEWER RODS

Flexible Sewer-Rod Eqpt. Co., 9059 Venice Blvd., Los Angeles, Calif.
W. H. Stewart, P. O. Box 767, Syracuse, N. Y.

SHOCK ABSORBERS

(for pipe lines)
Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio

SILENCERS, ENGINE, INTAKE, EXHAUST AND AIR INTAKE

Burgess-Manning Co., 2915 W. Roscoe, Chicago, Ill.

SIPHONS (Sewage)

American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City. (Alternating.)
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

SLEEVES, PIPE REPAIR

Clow & Sons, James B., 201 N. Talman Ave., Chicago, Ill.
Dresser Mfg. Div., Bradford, Pa.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Lee Hydraulic Co., Troy, N. Y.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
Warren Foundry & Pipe Corp., 11 Broadway, New York City.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

SLEEVES, TAPPING (Valve)

Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Lee Hydraulic Co., Troy, N. Y.
Ludlow Valve Mfg., Co., Troy, N. Y.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Rensselaer Valve Co., Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
Vogt Brothers Mfg. Co., Louisville, Ky.

SLUDGE COLLECTORS

American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dorr Co., 570 Lexington Ave., New York City.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Lakeside Engrg. Corp., 222 West Adams St., Chicago, Ill.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia, Pa.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Walker Process Equip., Inc., Aurora, Ill.
Yeomans Brothers Co., 1433 N. Dayton St., Chicago 22, Ill.

SLUDGE DISINTEGRATORS

Jeffrey Mfg. Co., Columbus, Ohio.
Royer Foundry & Mach. Co., 158 Pringle St., Kingston, Pa.

SLUDGE EJECTORS

(See Ejectors, Pneumatic.)

SLUDGE ELEVATORS, LIQUID

Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Hardinge Corp., York, Pa.

SLUDGE GAS ENGINES

(See Engines, Sludge Gas)

SLUDGE PUMPS

(See Pumps, Sludge)

SLUDGE SAMPLERS

Drake Mfg. Co., 1461 Belfield Ave., Philadelphia, Pa.

SNUBBERS

(See Silencers.)

SODA ASH

Diamond Alkali Co., Pittsburgh, Pa.
Innis, Speiden & Co., 117 Liberty St., New York City.
Mathieson Alkali Works, 60 E. 42nd St., New York City.
Niagara Alkali Co., 60 E. 42nd St., New York City.
Penn. Salt. Mfg. Co., Widener Bldg., Philadelphia, Pa.
Pittsburgh Plate Glass Co., Columbia Chemical Div., Grant Bldg., Pittsburgh 19, Pa.
E. M. Sergeant Pulp & Chemical Co., Inc., 7 Dey St., New York 7, N. Y.
Solvay Sales Corp., 40 Rector St., New York City.

SODIUM ALUMINATE

Dearborn Chemical Co., 310 Michigan Ave., Chicago, Ill.
Penn. Salt Mfg. Co., Widener Bldg., Philadelphia 7, Pa.

SODIUM BISULPHITE

General Chemical Co., 40 Rector St., New York, N. Y.

SODIUM CHLORITE

Mathieson Alkali Works, 60 East 42nd St., New York 17, N. Y.

SODIUM PHOSPHATES (Glassy)

Calgon, Inc., Hagan Bldg., Pittsburgh, Pa.

SODIUM SILICATE

Diamond Alkali Co., Pittsburgh, Pa.
General Chemical Co., 40 Rector St., New York 6, N. Y.
Philadelphia Quartz Co., Third & Ionic Sts., Philadelphia 6, Pa.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Standard Silicate Division, Diamond Alkali Co., Pittsburgh, Pa.

SOFTENERS

(See Water Softening Equipment)

SPEED REDUCERS, TRANSMISSION

De Laval Steam Turbine Co., Trenton, N. J.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Worthington Pump & Mach. Corp., Harrison, N. J.

SPRINKLING FILTER EQUIP.

American Well Works, Aurora, Ill.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dorr Co., 570 Lexington Ave., New York City.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Walker Process Equip., Inc., Aurora, Ill.
Yeomans Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

SPRINKLING FILTER UNDERDRAINS

(See Filter Underdrains, Sewage.)

STACKS

Cement Gun Co., P. O. Box 424, Allentown, Pa.
Chicago Bridge & Iron Co., 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
General American Process Equipment, 420 Lexington Ave., New York 17, N. Y.
Johns-Manville Co., 22 E. 40th St., New York City.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

STANDPIPES

(See Tanks and Standpipes.)

STANDPIPE AND TANK PRESERVATION

(See Cathodic Protection Process.)

STERILIZING WATER MAINS

(See Water Main Sterilization)

STIRRERS (Laboratory)

Dorr Co., 570 Lexington Ave., New York City.
Omega Mach. Co., Inc., 9 Codding St., Providence 1, R. I.
Phipps & Bird, Inc., Sixth & Byrd Sts., Richmond, Va.

STOKERS

Fairbanks, Morse & Co., 600 S. Michigan Ave., Chicago 5, Ill.
Johnston & Jennings Co., 877 Addison Rd., Cleveland, Ohio.
Link-Belt Co., 2410 W. 18th St., Chicago 8, Ill.
Westinghouse Electric & Mfg. Co., P. O. Box 868, Pittsburgh 30, Pa.

STORAGE TANKS (See Standpipes and Tanks)

STRAINERS, SUCTION

Barrett, Haentjens & Co., Hazleton, Pa.
Edson Corp., 49 D St., South Boston, Mass.
Golden-Anderson Valve Spec. Co., 1829 Fulton Bldg., Pittsburgh, Pa.

SULPHATE OF ALUMINA

(See Filter Alum)

SULPHUR DIOXIDE (Liquid)

Virginia Smelting Co., West Norfolk, Va.

SUPERNATANT LIQUOR (Selectors and Treaters)

Pacific Flush-Tank Co., 4241 Ravenswood Ave., Chicago, Ill.

SWITCHBOARDS & SWITCH GEAR

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Clark Controller Co., 1146 E. 152nd St., Cleveland 10, Ohio.
General Electric Co., Schenectady, N. Y.
Westinghouse Electric & Mfg. Co., P. O. Box 868, Pittsburgh 30, Pa.

TANK MAINTENANCE & REPAIR

Dixie Tank & Bridge Co., Lamar Ave., Memphis 1, Tenn.
Spielman Elevated Tanks Service, 822 N. Court St., Rockford, Ill.

TANKS AND STANDPIPES (Steel)

Chicago Bridge & Iron Co., 2198 McCormick Bldg., Chicago, Ill.
R. D. Cole Mfg. Co., Newnan, Ga.
General American Process Equipment Div., 420 Lexington Ave., New York 17, N. Y.
Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

TANKS, RUBBER LINED

Atlas Mineral Products Co., Mertztown, Pa.
Chemical Equipment Co., P. O. Box 3096, Terminal Annex, Los Angeles 12, Calif.
Dorr Co., 570 Lexington Ave., New York City.
Everson Mfg. Co., 214 W. Huron St., Chicago, Ill.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

TAPPING MACHINES

Glauber Brass Mfg. Co., Kinsman, Ohio.
Hays Mfg. Co., Erie, Pa.
Mueller Co., Decatur, Ill.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.

TASTE & ODOR REMOVAL CHEMICALS

Activated Alum Corp., Curtis Bay, Baltimore, Md.
General Chemical Co., 40 Rector St., New York 6, N. Y.
Industrial Chemical Sales Div., West Virginia Pulp & Paper Co., Inc., 230 Park Ave., New York 17, N. Y.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Mathieson Alkali Works, Inc., New York, N. Y.
Phelps Dodge Refg. Corp., 40 Wall St., New York, N. Y.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.
Virginia Smelting Co., West Norfolk, Va.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

TESTERS (Water Meter)

Ford Meter Box Co., Wabash, Ind.
Neptune Meter Co., 50 W. 50th St., New York City.
Northrop & Co., Inc., Spring Valley, N. Y.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.

Pittsburgh Equitable Meter Co., 400 Lexington Ave., Pittsburgh, Pa.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
Worthington-Gamon Meter Co., Newark, N. J.

TEST PLUGS (for Pipe)

Glauber Brass Mfg. Co., Kinsman, Ohio.
Richmond Fdry. & Mfg. Co., 1200 Hermitage Rd., Richmond, Va.

THICKENERS (Sludge)

Ralph B. Carter Co., 53 Park Place, New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Dorr Co., 570 Lexington Ave., New York City.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Yeomans Bros. Co., 1433 N. Dayton St., Chicago 22, Ill.

TOOLS, PIPE

(See Pipe Tools.)

TOOLS, PNEUMATIC

Davey Compressor Co., Kent, Ohio.
Jaeger Machine Co., 716 Dublin Ave., Columbus, Ohio.

TRANSFORMERS, ELECTRIC

General Electric Co., Schenectady, N. Y.
Westinghouse Electric Corp., Pittsburgh, Pa.

TRAPS, DRIP, FLAME, SEDIMENT

Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio.
Pacific Flush Tank Co., 4841 Ravenswood Ave., Chicago, Ill.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

TRENCHING EQUIPMENT

Hydrauger Corp., Ltd., 116 New Montgomery St., San Francisco, Calif.

TRICKLING FILTER UNDERDRAINS

(See Filter Underdrains.)

TRITURATORS

(See also Grinders)
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.

TUNNEL LINER PLATES

Armco Drainage Prod. Assn., Middletown, Ohio.

TUNNEL MACHINERY, BORING & LINING

Jeffrey Mfg. Co., Columbus, Ohio.

TURBINES, HYDRAULIC

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
De Laval Steam Turbine Co., Trenton, N. J.
Morris Machine Works, Baldwinsville, N. Y.
S. Morgan Smith Co., York, Pa.

TURBINES, STEAM

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
De Laval Steam Turbine Co., Trenton, N. J.
Morris Machine Works, Baldwinsville, N. Y.
Worthington Pump & Mach. Corp., Harrison, N. J.

UNIT SUBSTATIONS

General Electric Co., Schenectady, N. Y.
Westinghouse Electric Corp., Pittsburgh, Pa.

VALVE BOXES

Central Foundry Co., 386 Fourth Ave., New York City.
Chapman Valve Mfg., Indian Orchard, Mass.
Eddy Valve Co., Waterford, N. Y.
Ford Meter Box Co., Wabash, Ind.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Lee Hydraulic Co., Troy, N. Y.
Ludlow Valve Mfg. Co., Troy, N. Y.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVE BOX LOCATORS

Northrop & Co., Inc., (Spring Valley, N. Y.)

VALVE OPERATOR

(See Gate Valve Operator)

VALVES

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Anderson Valve Corp., Oliver Bldg., Pittsburgh, Pa.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Eddy Valve Co., Waterford, N. Y.
Golden-Anderson Valve Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
M. & H. Valve & Fittings Co., Anniston, Ala.
Merco Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.
Mueller Co., Chattanooga, Tenn.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
R-S Products Corp., Philadelphia 44, Pa.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
S. Morgan Smith Co., York, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.
Vogt Bros. Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, AIR RELEASE

Chapman Valve Mfg. Co., Indian Orchard, Mass.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
Valve and Primer Corp., Chicago 2, Ill.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, AIR & VACUUM

Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Roots-Connersville Blower Corp., Connersville, Ind.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
Valve and Primer Corp., 30 W. Washington St., Chicago, Ill.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, ALTITUDE

Anderson Valve Corp., Oliver Bldg., Pittsburgh, Pa.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
R-S Products Corp., Philadelphia 44, Pa.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
S. Morgan Smith Co., York, Pa.
Valve and Primer Corp., 30 W. Washington St., Chicago, Ill.

VALVES, BACKWATER

Josam Mfg. Co., 1783 E. 11th St., Cleveland, O.

VALVES, BUTTERFLY

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Ludlow Valve Mfg. Co., Troy, N. Y.
R-S Products Corp., Philadelphia 44, Pa.

VALVES, CHECK

Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Barrett, Haentjens & Co., Hazleton, Pa.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Darling Valve Co., Williamsport, Pa.
Eddy Valve Co., Waterford, N. Y.
Golden-Anderson Valve Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio
Ludlow Valve Mfg. Co., Troy, N. Y.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
S. Morgan Smith Co., York, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, CHLORINE

Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Everson Filter Service Co., 214 W. Huron St., Chicago, Ill.
Wallace & Tiernan Co., Inc., Newark 1, N. J.

VALVES, CONE

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Merco Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
S. Morgan Smith Co., York, Pa.

VALVES, DIAPHRAGM

McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.

VALVES, ELECTRICALLY OPERATED

(See Valves, Motor Operated.)

VALVES, EMERGENCY TRIP

Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Ross Valve Mfg. Co., Troy, N. Y.

VALVES, FLAP

Chapman Valve Mfg. Co., Indian Orchard, Mass.
Eddy Valve Co., Waterford, N. Y.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio.
Ludlow Valve Mfg., Co., Troy, N. Y.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Rensselaer Valve Co., Troy, N. Y.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, FLOAT

Chapman Valve Mfg. Co., Indian Orchard, Mass.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
S. Morgan Smith Co., York, Pa.

VALVES, FOOT

Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill
Eddy Valve Co., Waterford, N. Y.
Edson Corp., 49 D St., South Boston, Mass.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
Mueller Co., Chattanooga, Tenn.
Northrop & Co., Inc., Spring Valley, N. Y.
Rensselaer Valve Co., Troy, N. Y.
Vogt Bros. Mfg. Co., 1402 W. Main St., Louisville 3, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, FOUR WAY

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Merco Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
R-S Products Corp., Philadelphia, Pa.
Roberts Filter Mfg. Co., Darby, Pa.

VALVES, GATE

American Valve Mfg. Co., 37-25 32nd St., Long Island City 1, N. Y.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Darling Valve Co., Williamsport, Pa.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Rensselaer Valve Co., Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, HYDRAULIC CYLINDER OPERATED

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Eddy Valve Co., Waterford, N. Y.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.

M. & H. Valve & Fittings Co., Anniston, Ala.
Merco Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.
Mueller Co., Chattanooga, Tenn.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Rensselaer Valve Co., Troy N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
S. Morgan Smith Co., York, Pa.
Vogt Brothers Mfg. Co., Louisville, Ky.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, LUBRICATED PLUG

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Merco-Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.

VALVES, MOTOR OPERATED

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Eddy Valve Co., Waterford, N. Y.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Merco Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.
Mueller Co., Chattanooga, Tenn.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., East Orange, N. J.
S. Morgan Smith Co., York, Pa.
R. D. Wood Co., Public Ledger Bldg., Independence Square, Philadelphia, Pa.

VALVES, MUD AND PLUG

American Car & Fdy. Co., 30 Church St., New York 8, N. Y.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Eddy Valve Co., Waterford, N. Y.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Ludlow Valve Mfg. Co., Troy, N. Y.
Merco Nordstrom Valve Co., 400 N. Lexington Ave., Pittsburgh, Pa.
M. & H. Valve & Fittings Co., Anniston, Ala.
Mueller Co., Chattanooga, Tenn.
Peerless Pump Div., Food Machinery Corp., 301 W. Avenue 26, Los Angeles 31, Calif.
Rensselaer Valve Co., Troy, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Vogt Brothers Mfg. Co., Louisville, Ky.

VALVES, PRESSURE REDUCING

Automatic Control Div. Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Hays Mfg. Co., Erie, Pa.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
S. Morgan Smith, York, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

VALVES, RELIEF

Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Chapman Valve Mfg. Co., Indian Orchard, Mass.
Crane Co., 836 S. Michigan Ave., Chicago, Ill.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Hays Mfg. Co., Erie, Pa.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Ludlow Valve Mfg. Co., Troy, N. Y.
Mueller Co., Decatur, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Rensselaer Valve Co., Troy, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
S. Morgan Smith Co., York, Pa.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

VALVES, RUBBER, GLASS AND LEAD LINED

American Hard Rubber Co., 11 Mercer St., New York 13, N. Y.
Automatic Control Div., Climax Industries, 1949 S. Western Ave., Chicago, Ill.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Grinnell Co., Inc., 260 W. Exchange St., Providence, R. I.
Iowa Valve Co., Hubbell Bldg., Des Moines, Ia.
Johns-Manville, 22 E. 40th St., New York, N. Y.
S. Morgan Smith, York, Pa.

VALVES, SPECIAL, AUTOMATIC (Including Remote Control.)

Barrett, Haentjens & Co., Hazleton, Pa.
Golden-Anderson Valve Spec. Co., 1329 Fulton Bldg., Pittsburgh, Pa.
Iowa Valve Co., Hubbell Bldg., Des Moines, Iowa.
McAlear Mfg. Co., 1949 S. Western Ave., Chicago 8, Ill.
Northrop & Co., Inc., Spring Valley, N. Y.
Ross Valve Mfg. Co., P. O. Box 595, Troy, N. Y.
Rensselaer Valve Co., Troy, N. Y.
S. Morgan Smith Co., York, Pa.
A. P. Smith Mfg. Co., 545 N. Arlington Ave., E. Orange, N. J.

VALVES, WATER METER SPECIAL

Ford Meter Box Co., Wabash, Ind.

VARIABLE SPEED TRANSMISSIONS

Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.
Omega Machine Co., 9 Codding St., Providence 1, R. I.

VENTURI TUBES

Bailey Meter Co., 1072 Ivanhoe, Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St. Providence, R. I.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Leupold & Stevens Instruments, 4445 N. E. Glisan St., Portland, Ore.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.

VIBRATING SCREENS (Liquid)

Link Belt Co., 307 N. Michigan Ave., Chicago 1, Ill.

VIBRATORS, ELECTRO-MAGNETIC

Jeffrey Mfg. Co., Columbus, Ohio.
Link-Belt Co., 2045 W. Hunting Park Ave., Philadelphia 40, Pa.

VISES, PIPE

Ridge Tool Co., Elyria, Ohio.

VITRIFIED CLAY PIPE AND PRODUCTS

Nat'l Clay Pipe Mfrs., Inc., 111 W. Washington St., Chicago, Ill.

WASTE GAS BURNERS

Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Vapor Recovery Systems Co., 2820 N. Alameda St., Compton, Calif.

WATER ANALYSIS EQUIPMENT AND REAGENTS

(See Laboratory Apparatus and Reagent.)

WATER HAMMER ARRESTERS

Josam Mfg. Co., 1783 E. 11th St., Cleveland, Ohio.
S. Morgan Smith Co., York, Pa.

WATER MAIN STERILIZATION (A Service)

Wood and Preuit, 1740 K St., N.W., Washington, D. C.

WATER LEAK, LOCATING INSTRUMENTS

Fisher Research Lab., 1961-63-65 University Ave., Palo Alto, Calif.
Globe Phone Mfg. Co., Reading, Mass.
Northrop & Co., Inc., Spring Valley, N. Y.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.

WATER LEVEL CONTROL

Automatic Control Corp., 1013 University Ave., St. Paul 4, Minn.
B/W Controller Corp., Birmingham, Mich.

WATER LEVEL RECORDERS

Bailey Meter Co., 1072 Ivanhoe Rd., Cleveland, Ohio.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
R. W. Sparling, Box 3277 Terminal Annex, Los Angeles, Calif.

WATER MAIN CLEANING

Flexible Underground Pipe Cleaning Co., 9059 Venice Blvd., Los Angeles, Calif.
National Water Main Cleaning Co., 30 Church St., New York 7, N. Y.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.
Geo. H. Werfelman Co., Los Angeles, Cal.

WATERPROOFING PAINTS, COMPOUNDS AND MATERIALS

General Paint Corp., Cleveland, O.
Inertol Co., Newark 5, N. Y.
Johns-Manville, 22 E. 40th St., New York City.
Wailes Dove-Hermiston, Westfield, N. J.

WATER SOFTENING CHEMICALS AND COMPOUNDS

(See Individual Chemicals for Water Purification and Softening)

WATER SOFTENING EQUIPMENT

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
American Well Works, Aurora, Ill.
Builders-Providence, Inc., 9 Codding St., Providence, R. I.
Ralph B. Carter Co., 53 Park Pl., New York City.
Chain Belt Co., 1610 W. Bruce St., Milwaukee 4, Wis.
Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Derr Co., Inc., 570 Lexington Ave., New York City.
Everson Mfg., Co., 214 W. Huron St., Chicago, Ill.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Jeffrey Mfg. Co., Columbus, Ohio.
Lakeside Engrg. Corp., 222 W. Adams St., Chicago, Ill.
F. B. Leopold Co., Inc., 421 First Ave., Pittsburgh, Pa.
Link Belt Co., 2045 W. Hunting Park Ave., Philadelphia, Pa.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Omega Machine Co., 9 Codding St., Providence, R. I.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Proportioneers, Inc., 9 Codding St., Providence, R. I.
Roberts Filter Mfg. Co., Darby, Pa.
Simplex Valve & Meter Co., 68th and Upland Sts., Philadelphia, Pa.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.
Walker Process Equip., Inc., Aurora, Ill.
Wallace & Tiernan Co., Inc., Newark, N. J.
Worthington Pump & Mach. Corp., Harrison, N. J.
Zeolite Chemical Co., 140 Cedar St., New York 6, N. Y.

WATER STABILIZATION (Chemicals for)

Calgon, Inc., Hagan Bldg., Pittsburgh, Pa.

WATER SUPPLY CONTRACTORS

(See Contractors)

WATER TREATMENT FOR SCALE AND CORROSION

Allis-Chalmers Mfg. Co., Milwaukee 1, Wis.
Calgon, Inc., Hagan Bldg., Pittsburgh, Pa.
General Chemical Co., 40 Rector St., New York 6, N. Y.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Stuart Corp., 516 N. Charles St., Baltimore 1, Md.
Zeolite Chemical Co., 140 Cedar St., New York 6, N. Y.

WATER TREATMENT PLANTS (Steel)

Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa.

WATER WASTE SURVEYS

Pitometer Co., 50 Church St., New York City.
Simplex Valve & Meter Co., 68th & Upland Sts., Philadelphia, Pa.

WEED KILLERS

C. B. Dolge Co., Westport, Conn.

WELLS, COMPLETE

Layne-Bowler, Inc., P. O. Box 215, Hollywood Sta., Memphis 8, Tenn.

WIND VANES AND INDICATORS

M. C. Stewart, Ashburnham, Mass.

WRAPPING, PIPE

(See Pipe, Wrapping)

WRENCHES, REVERSIBLE RATCHET

Dresser Mfg. Co., Bradford, Pa.

WROUGHT IRON (Pipe, Gates, Bars, Etc.)

A. M. Byers Co., Clark Bldg., Pittsburgh, Pa.

ZEOLITE AND ZEOLITE SOFTENERS

Chemical Equipment Co., P. O. Box 3098, Terminal Annex, Los Angeles 12, Calif.
Infilco, Inc., 325 W. 25th Place, Chicago, Ill.
Liquid Conditioning Corp., 114 E. Price St., Linden, N. J.
Permutit Co., 330 W. 42nd St., New York 18, N. Y.
Roberts Filter Mfg. Co., Darby, Pa.
Worthington Pump & Mach. Corp., Harrison, N. J.
Zeolite Chemical Co., 140 Cedar St., New York 6, N. Y.

HYDRAULICS FOR THE PRACTICAL OPERATOR

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SECTION I

General Principles

THE science of hydraulics, which deals with the flow of fluids in pipes, channels, pumps, turbines and other machines and devices, has been looked upon by many operators of waterworks plants and by engineers without college training as something formidable, generally because they have felt that its principles and ideas offered mathematical difficulties beyond their grasp. They have thus denied themselves the use of a tool which would enable them to solve many everyday problems arising in their work.

Actually the subject presents few theoretical difficulties, and anyone knowing the algebra of the first year in high school would have no difficulty with fairly advanced work in hydraulics, and a man with a good knowledge of arithmetic should be able to solve most problems that arise. The usual trouble, where it exists, is not with the technical side but with the application of the principles involved, and in this respect the practical engineer has many advantages, for he can visualize the conditions as they arise; without his background a full understanding of this subject is not very easy.

Hydraulics is used in solving problems on flow of all fluids whether they are liquids (incompressible) or compressible gases such as air and chimney gases, or vapors such as steam, but at present, consideration will be limited to liquids, of which the most common is water. In the daily routine of water and sewage works operations the flow of oils and miscellaneous liquids forms a very small part and, in any event, nearly all such problems may be solved by the same methods as apply to water.

In this article the author will try to lay down in simplest form the basic principles underlying the flow of water and will show their application in a number of practical problems.

One Important General Principle

In establishing the one general principle which occurs perpetually in this work, the usual practice is to proceed by considering the power carried by the water in its passage along a pipe, canal, or machine. Care must, of course, be taken with the units employed, as much confusion and error results from neglect in this respect. Lengths are measured in feet, areas in square feet, and discharges in United States gallons per minute. Where the Imperial gallon is used, as in Canada and Great Britain, it is very easy to get the equivalent in United States gallons by remembering that one Imperial gallon has the same volume as 1.2 United States gallons. The United States gallon of cold water at ordinary temperature weighs 8.34 lbs., therefore the Imperial gallon weighs $8.34 \times 1.2 = 10$ lbs., and 1 cubic foot of water under the same conditions weighs 62.4 lbs. These figures occur very often and must be remembered or noted for easy reference.

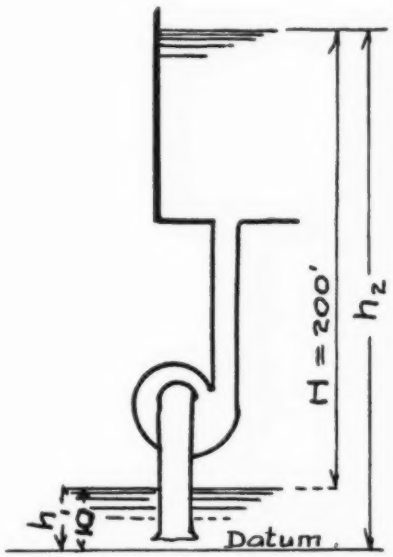


Fig. 1—Head and Power Relationships.

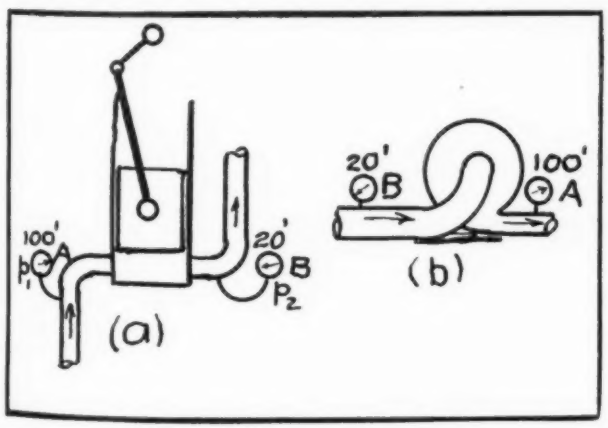
Practically all hydraulic problems involve the velocity of water in the pipe or channel, and indeed speed is one of the most important considerations in this and almost every other science. Speed determines the type of pumping engines used, the general shape of locomotives and railroad cars, the choice between centrifugal and reciprocating pumps, etc. High speed machines are usually relatively small and compact, just as pipes carrying water at high velocity are small, but with the high speed, wear and losses of power occur and, therefore, convenience and size have to be balanced over against the disadvantages.

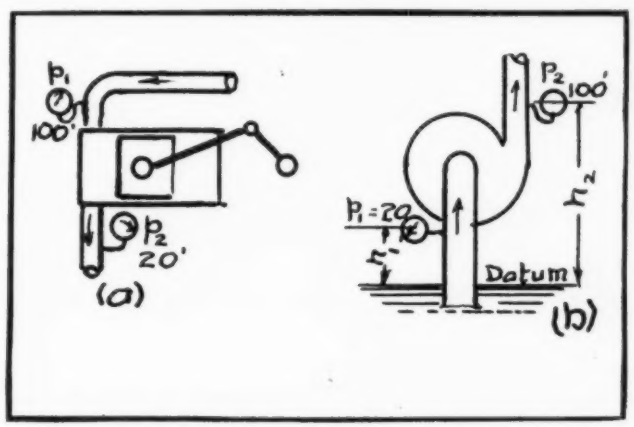
In waterworks piping particularly, fairly low velocities are common, because the pipes are long and the losses must be reduced, and also because the cost of laying such pipes is high relative to the cost of the pipes, so that it is best to have them on the large side. Velocities of 4 or 5 ft. per sec. are not uncommon, and during fires they may go much higher, but velocities below those named are not used except with intakes and such work where loss must be kept to a minimum, or else where a pipe supplies a growing district. For convenient reference, Table I has been made. This table gives velocities in pipes up to 12 in. diameter for different flows, and this table may be easily used for larger pipes and other discharges. For example, although a 20 in. pipe has a diameter only twice that of a 10 in., the area is four times as great. Therefore, the velocity in the 20 in. pipe for a given discharge is only one-quarter that in the 10 in. for the same discharge.

In setting up the basic laws and relationships that must be employed, the usual and simplest method is to consider the power carried in the water. Power in flowing fluids exists in several forms; for example, where a pump delivers water from a well to a reservoir it has lifted the water from a low elevation to a higher one, and for that effect each pound of water had work done on it by the pump.

The difference in levels of the water surfaces is commonly called the "head" on the pump in feet, this head simply being the difference in elevation of each pound of water as it is pumped, that is, each pound of water has changed its "elevation head."

The power required to produce this change in elevation is the weight of water per minute multiplied by this change in elevation, or if the pump delivers G gallons per min., the power in ft. lbs. per min. will be 8.34 G times the change in elevation. Since there are 33,000 ft. lbs. per min. corresponding to a horsepower, the hp. put into the water by above pump would be 8.34 G





Figs. 2 and 3—Comparing Water Driven Engines and Centrifugal Pumps Under Different Conditions

times change in elevation ÷ 33,000 into the water; naturally, the power on the pump shaft would exceed this by the losses in the pump and piping. To raise 1,200 gal. per min. a vertical distance of 200 ft. from well to reservoir level would require 8.34 × 1,200 × 200 ÷ 33,000 = 60.6 H.P. put into the water or a shaft horsepower perhaps over 80.

The diagram of Fig. 1 illustrates this case and the head on the pump is H ft., but it is well at this point to mention a method in very common use to simplify the arithmetic of the problem. Usually all heights are measured from a "datum plane" or "datum" selected at any convenient level, but always below the lowest part of the pipe or other device being studied. By this method all vertical distances are measured upward from the datum plane and thus one avoids the necessity of using the words "above" and "below" or signs plus and minus to indicate the direction in which the measurement is made. In this figure the datum plane has been chosen 10 ft. below the surface of the well so as to be sure to be below the bottom of the suction pipe, but it will be noted that the difference in levels of the water surfaces is the same no matter where this datum is placed. In Fig. 1 the elevation heads of the well and the reservoir water surfaces are represented by $h_1 = 10$ ft. and $h_2 = 210$ ft., and hence $h_2 - h_1 = 200$ ft.

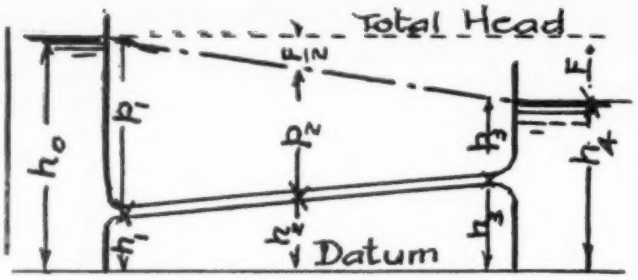


Fig. 4—Elevation and Pressure Heads

But water flowing in pipes contains power in other forms than that due to elevation because often power is transmitted in a machine by or to water without any change in elevation taking place; this statement is true in most engines operated by water and in many centrifugal pumps, the power in these cases being due to a change in pressure of the water in the machine. This is illustrated in Fig. 2, where a water driven engine is shown at (a) and a centrifugal pump at (b), and in both cases it is assumed that the water enters and leaves through pipes of the same size and at the same level.

Gages at A and B on the pipe indicate the pressures, which may be in lb. per sq. in. or in feet of water, as the gages are sometimes graduated in one unit and sometimes in the other, but rarely in both. If both graduations are used it will be found that 231 ft. of water is also marked 100 lb. per sq. in. and this ratio of 2.31 ft. of water = 1 lb. per sq. in. pressure is the same all over the scale, but this matter will be discussed later.

TABLE I
Velocities in Pipes

Pipe size (in.)	Actual diam. (in.)	Area (sq. ft.)	Velocity ft. per sec. for 100 U. S. gal. per min.	Pipe size (in.)	Area (sq. ft.)	Velocity ft. per sec. for 1,000 U. S. gal. per min.
1	1.049	.00600	37.1	4	.0873	25.50
1¼	1.25	.00852	26.2	6	.1963	11.33
1½	1.50	.01227	18.2	8	.349	6.37
2	2.067	.0233	9.6	10	.545	4.07
3	3.068	.0513	4.3	12	.785	2.83

1,000 U. S. gallons per minute = 1.44 million gallons per 24 hours. One cubic foot per second (1 cfs.) = 450 U. S. gallons per minute (450 gpm).

At present it will be assumed that the graduations are in feet of water and they mean that if, for example, the gage is attached to the base of a standpipe in which the water surface is 60 ft. above the gage, then the latter would show 60 ft. if the gage was in proper condition. In the case of Fig. 2(a), suppose gage A shows 100 ft. and gage B shows 20 ft., then the effect is the same as if the engine was attached at A to a tank in which the water level was 100 ft. above it and at B to another tank in which the water level was 20 ft. above it. Then the effective head on the engine is 100 − 20 = 80 ft. in exactly the same way as if elevation head of the same magnitude was available. These pressures in feet, read from the gages, are called "pressure heads" and the power produced is calculated in exactly the same way as it would be for 80 ft. difference in elevation heads. The case for the pump is exactly the same as for the engine. One thing should be reiterated here, however, and that is that the inlet and outlet pipes have been assumed the same size and condition and no effect of differences of velocity need be considered. As a matter of fact, the water surface in the assumed tank at A for the engine shown at (a) would need to be more than 100 ft. above the gage because velocity and friction have to be considered, but in practice the effect of these is so small that at this stage they may be neglected in this case.

It often happens, however, that the inlet and outlet pipes are not at the same level. Such an instance is illustrated in Fig. 3 where the difference of levels of the gages is taken as 4 ft. If the gages in this case read 100 ft. and 20 ft., respectively, then in case (a) the engine not only gets the effect of the difference of pressure of 80 ft., but it also gets the benefit of the difference of elevation of 4 ft. and hence the effective head is the sum of the net pressure head and the change in elevation (or the elevation head), a similar statement being equally true of the pump. Therefore, to obtain the effective head acting at any point on the pipe, the elevation head is to be added to the pressure head. This may be illustrated very nicely in case of the pipe shown in Fig. 4, where the elevation heads are marked h_1 and h_2 and h_3 and the pressure heads p_1 and p_2 and p_3, and these are plotted to scale, but anyone who does not like the elegant graphical representation may, of course, just carry out the work by arithmetic. All inlet and outlet pipes are assumed in this case to be the same size, variations in these being left for later consideration.

Interpretation of Gage Readings

In finding pressures at any section under consideration, care must be taken in interpreting gage readings. The gage shows the pressure in the spring tube inside it and this may be the same as or different from that in the pipe at the point where the gage is connected. To illustrate this, imagine a standpipe filled to a depth of 60 ft. with water; the pressure at its bottom is 60 ft. and half way up it is 30 ft., and so on. If a gage connection is made in the side of the standpipe half way up and a gage attached so that its center is opposite the connection, such a gage will read 30 ft. of water; but if there is a run of small pipe running from the gage to the above connection and it is of

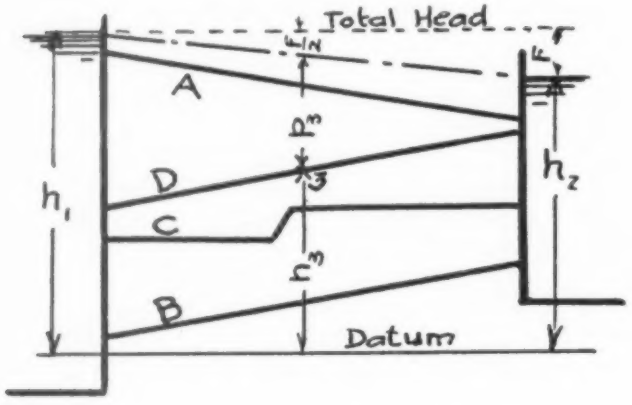


Fig. 5—Behavior of Pipes and Friction Effects

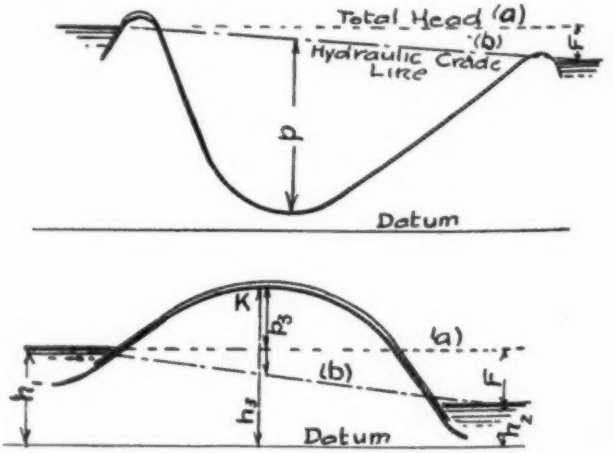


Fig. 6—Hydraulic Grade Lines and a Syphon

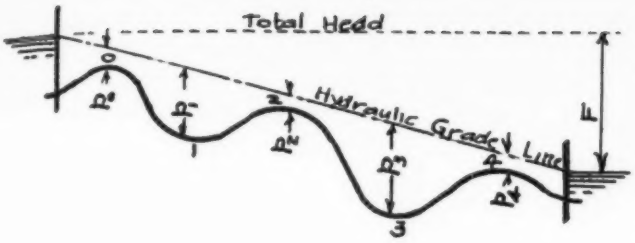


Fig. 7—Importance of the Hydraulic Grade Line and Pipe Line Design

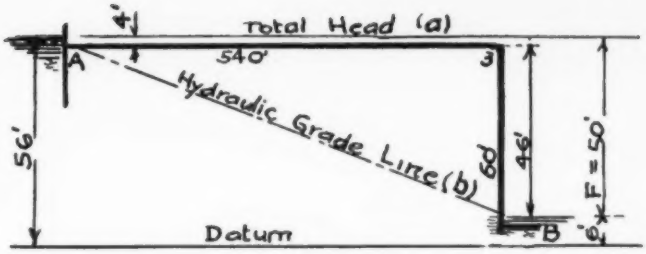


Fig. 8—Faulty Pipe Line Design and Poor Performance

such a length that the center of the gage attached to it is 10 ft. below the connection, then the gage will read 40 ft. of water; similarly, if the run of the small pipe is such that the center of the gage is 10 ft. above the point of connection, the gage will read only 20 ft. of water.

Where there is pressure greater than atmospheric it is necessary to correct the gage reading by adding the vertical distance in feet that the gage is above the point of pipe connection, or if the gage is below the connection, this vertical distance must be subtracted. The drawings are intended to indicate corrected pressures. This is done because the pressure due to the weight of water in the gage connecting pipe must be taken into account. If, however, there is a vacuum in the pipe to which the gage connection is made, as, for example, in the suction pipes of many pumps, and if care is taken to free the small pipe between the gage connection and the gage of all water, then the gage will read correctly the pressure at the point where the connection is made to the pipe, regardless of whether the gage is at the same level as this connection or not.

If a gage connected directly (without intermediate piping) to a suction pipe should show a vacuum of 20 ft., this same gage would also read practically 20 ft. if it was lowered 10 ft. by using long piping free of water between the connection and the gage. Actually, there would be a slight difference due to the weight of the air in the small gage piping, but this would be insignificant. If, however, the gage piping becomes filled with water all the way from the gage to the suction piping, then lowering the gage 10 ft. would raise the pressure recorded by it by the same amount, that is, it would show a vacuum of only 10 ft. For vacuum gages it is good practice, therefore, to blow all the water out of the gage connection piping by opening, for a few seconds, a petcock connected to such piping close to the gage, and this cock should be opened at times during the day, as water may collect there.

Unless attention is paid to these points, quite serious misunderstanding of the gage readings may result. For instance, if as in the last case mentioned above, the gage connecting piping gradually fills with water, it is evident that the vacuum may change from an actual 20 ft. to an apparent 10 ft. Wherever possible, the gage connection should be short and both air and water tight, and it is best to attach the gage on the same level as the connection, but the latter point is not so serious. If the gage connection piping on a pressure gage leaks, errors often result in the gage indications because there is a definite pressure drop along the gage piping, but care must be taken that the connecting pipe is full of water and it should occasionally be blown out by opening the petcock near the gage so as to clear air from the gage pipe.

Friction and Resistance

Suppose now that gages are attached at some distance apart on a horizontal pipe of uniform size, then from previous discussion the gages should show the same pressure if there is no loss of power between them, but every practical man knows that with water flowing through the pipe the downstream gage always reads the lower of the two because of the known effect of friction of the water on the walls of the pipe. The difference in pressure increases as the distance between the gage connections increases, as the flow increases, and as the pipe gets older because it gets rusty and rough inside.

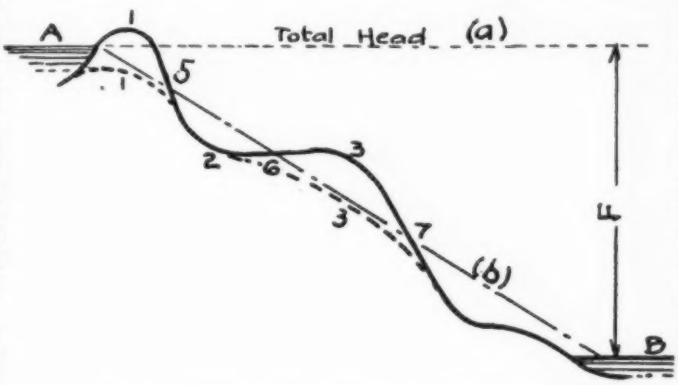


Fig. 9—Gravity Main Subject to Giving Trouble

The whole question of friction will be considered by itself because of its great importance, but at present it is only necessary to call attention to its existence. The head lost in this way will be designated by **F** ft. and, for example, a centrifugal pump would have to lift the water from the well level to the reservoir level and, in addition, overcome the friction head **F** ft.

An interesting illustration is an intake pipe connecting a lake with a pump well on shore, and here observation shows that the water in the well is lower than the lake; briefly, the water will not flow in there if it is not. Now, in the well and in the lake a pound of water is only capable of doing work because of its elevation, and if a pound at the surface is considered in each case, since there is no pressure on it, this difference of levels must correspond to the motive force causing flow. But since the only loss between the lake and the well is in the pipe, then the difference of levels must represent the friction head **F** ft.; the more water that is drawn the greater the difference in the two levels, as is well known, because the pipe losses increase.

The loss of head in friction will evidently remain constant for a certain length of pipe of any given size and fixed discharge, that is to say, it is not affected by the location of the pipe. If an intake consists of 3,000 ft. of 30 in. pipe the loss in this pipe for a certain flow will be the same whether the pipe lies on the bed of the lake or 10 ft. above or 3 ft. below the bed and will be the same whether the pipe is horizontal or slopes upward or downward. The actuating force is proportional to the difference in levels between the lake and well or to the difference in their elevation heads. As this point is important, it will be desirable to dwell on it further. In Fig. 5 two open ponds are shown connected by four pipes which carry water from the upper to the lower pond. For the time being the effect of the velocity of the water will be left out of account; in most cases it is relatively small.

Now let it be assumed that the levels of the ponds do not change or that h_1-h_2 has a fixed value; then if the four pipes **A, B, C** and **D** are all the same length, diameter and internal smoothness, the discharge will be the same for each of the pipes, because under the same discharge the friction loss ($F = h_1 - h_2$) will be the same in each case. In the upper pond the only possible source of power is the elevation head h_1 and the horizontal dotted line extended from the upper water level will therefore represent the total head, **F**. At the mid-point 3 on the pipe **D** the friction loss is clearly ½ **F** and, therefore, at this point the elevation head plus the pressure head plus the friction loss (½ **F**) must make up h_1 if the effect of velocity is not considered for the moment. For the pipe **D** the elevation head is h_3 and hence the pressure head is p_3 as marked on the figure, since $h_1 = h_3 + p_3 +$ ½ **F**. Clearly, the difference between the pipes **A** and **C** is that in the latter case all the pressures in the pipe are more than in the former, while for **B** the left hand end has much higher pressures than the right hand end of the same pipe.

Actually, it is always better, if possible, to have the pipe slope somewhat upward in the direction of flow as in the case of **B** and **D**, because air is sometimes present in the pipe and in case of **B** or **D** it would be carried through the pipe with the water, while in positions **A** and **C** it would be trapped there and would interfere with the flow.

A valley or a hill between the ponds may force the laying of the pipe in either of the positions shown in Fig. 6. For the valley in the upper figure the pressure in the pipe at the center is **p**, which is much higher than at other points and in the old Roman days when lead pipes of no great strength were available, the engineers had to carry

water over such a valley in an open aqueduct because a pipe laid as shown would have burst.

For the intervening hill in the lower diagram of Fig. 6 the elevation head h_3 is greater than h_1 and hence the only way it is possible to have $h_1 = h_3 + p_3 + \frac{1}{2} F$ is to have p_3 negative or less than atmospheric. In other words, there is a partial vacuum at **K** and the pipe line becomes a siphon. If the partial vacuum at **K** is not too high, the line will work, once it is started, as every engineer knows by experience. There are objections to siphons, and these will be discussed as the articles continue. If the reader will take an ordinary pad and draw the diagrams roughly to scale, it will be most helpful. He can use such scales as 8 ft. to 1 in. vertically and 800 ft. to 1 in. horizontally, so as to magnify the points he is examining. Incidentally, the pipe line running from one reservoir through a valley and up to another is known as an "inverted syphon." Actually it is no syphon at all.

The Hydraulic Gradient

If on this scale drawing, Fig. 6, a horizontal line of total head is plotted, as at (a), and below it a line corresponding to the friction losses, as at (b), the whole conditions in the pipe are shown graphically. The line (b) will be seen to be practically straight, since in a uniform pipe the friction loss increases with the length only; it starts at the upper surface and finishes at the lower one, and if the reader will only have patience and try to grasp the underlying principles, he will be shown later how the exact position of the line (b) is found.

The line (b) is commonly called the "hydraulic gradient" or hydraulic grade line. Wherever this line lies above the pipe there is pressure in the pipe, and where it is below the pipe there is a partial vacuum, and the pressure or vacuum in feet of water is just the vertical distance between the grade line and the pipe at that point. If velocity effect is included, as it should be, the pressure measured in the above way would, in a case of this kind, be reduced about half a foot, so that for the preliminary ideas velocity may be neglected.

These principles are most important in laying pipe and indicate that if the center line of the pipe is exactly on the line (b), or the "hydraulic gradient" line, there will be no pressure in the pipe, i.e., the water will be at atmospheric pressure. Since a perfect vacuum corresponds to a vacuum gage reading of 34 ft., it follows that the pipe must at every point be less than 34 ft. above (b). If the pipe is laid in uneven ground and rises and falls along its length but does not lie above the hydraulic grade line (b) at any point, as shown in Fig. 7, then although the pressure is always greater than atmospheric all along this pipe line, it is evident that points 0, 2 and 4 are at lower pressure than those at 1 and 3. As the water flows along the pipe it soon reaches a low pressure p_0, then a higher pressure p_1, then lower p_2, and so on.

All waters contain a certain volume of dissolved air, some much more than others. If the water is under high pressure the air is compressed and its volume is small, but as the water pressure is lowered the air expands and has a tendency to free itself from the water; such points as 0, 2 and 4 greatly facilitate this and experience shows that air actually comes out of the water and collects in the top of the pipe at such points; it will, of course, do the same thing and to greater degree at the highest point in a siphon. Since the pipe is commonly laid parallel to the surface of the ground, in all ordinary cases, it usually does have a shape

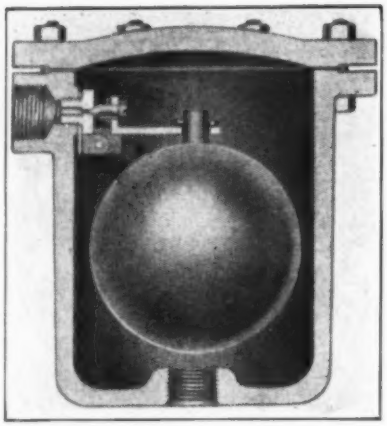


Fig. 10—Air Release Valve of Float Type

similar to that of Fig. 7 and experience has shown that some means must be provided for bleeding the air out of the pipe at the points 0, 2, 4, etc. Sometimes cocks are tapped into the top of the pipe at these places and men go along occasionally and open them to release the air.

In cities and other built-up areas it is obviously impossible to do this and automatic air valves are installed at these points, one type of such valve being shown at Fig. 10. They all work on the same general principle of having a valve similar to that on a household toilet, where a float opens and closes the valve. This float rests on the water surface in the pipe and as the latter fills with air at the high point the float descends and opens the valve, releasing the air. The water entering the air-bled float chamber causes the valve to close again. In case of a siphon, the pressure at the high point is less than atmospheric and a vacuum pump must be used to exhaust the air. Because this process is rather inconvenient and expensive, siphons are not used except in unusual cases.

The points just discussed lead to some very unexpected results and show why, in some cases, there is a failure of a pipe line or a pump suction line to deliver water. Fig. 8 shows a pipe connecting two tanks with a difference of 50 ft. in level; the pipe is a fairly long one, the first part being nearly level and the vertical drop of 50 ft. occurs close to the end of the line. In this case, the friction loss **F** is 50 ft. in the entire pipe, and since the end 3 **B** is short, as stated, most of the loss will take place between **A** and 3. As a definite case, take the length **A** 3 as 540 ft. and 3 **B** as 60 ft., making a total length of 600 ft., then the loss from **A** to 3 is 90 per cent of **F**, or 45 ft., and the loss in 3 **B** is 10 per cent of **F**, or 5 ft.

Taking the datum 6 ft. below the lower surface, partly for convenience, and drawing the total head line (a) through the upper surface and assuming the part **A** 3 of the pipe is laid 4 ft. below this line of total head, it will appear that at 3 the elevation head is 52 ft. and, since the friction loss is $F_3 = 45$ ft., the figure shows that the total head, 56 ft. above the selected datum, would, at point 3, equal the elevation head of 52 ft. plus the pressure head, plus the friction loss of 45 ft. That is $56 = 52 + p_3 + 45$ or $p_3 = 56 - 97 = -41$ ft.

This diagram will correspond to a vacuum of 41 ft. of water, but as 34 ft. of water is the highest vacuum possible, the pipe will not work satisfactorily and will only deliver spasmodically. The pipe would have to be lowered at least 7 ft. at 3 to avoid exceeding the 34 ft. negative head. The preferred position would, of course, be along or below the sloping hydraulic grade line (b).

Without unduly laboring this point, attention might be called to another practical case. In Fig. 9 a pipe is laid from a spring to a cistern or a tank in the house, and although following the contour of the land, it never slopes upward in its length after the start, as shown. Such pipe lines often give trouble, as this discussion would suggest. Drawing the line of total head (a) and the hydraulic grade line (b), the first of which is straight and horizontal and the latter nearly straight and sloping nearly evenly from the spring to the cistern, the pipe crosses the gradient **A B** at points 5, 6 and 7, at which places there is atmospheric pressure in the pipe. At points 1 and 3 there is partial vacuum, and as these are "high points" on the line, air will usually collect in the pipe and the latter may become "air bound" and refuse to work at all. The line is then actually air blocked or air locked.

Placing air cocks at 1 and 3 does no good because opening them would admit air and make matters worse, but if the pipe was laid in the lower dotted position (2, 3, 7) air would still collect at 1 and 3 but it could be released by opening air cocks at these points, since the pressures are both above that of the atmosphere. The somewhat distorted appearance of the drawing is deceptive for the horizontal scale is often 100 times as great as the vertical, so as to magnify the conditions. It would be very enlightening to plot on a plain sheet of paper, with an ordinary steel scale, a case like the above, first with the vertical scale and the horizontal scale, both 80 ft. to an inch, that is, each $\frac{1}{8}$ in. represents 10 ft., and then plot the same line with the same horizontal scale and with a vertical scale of 8 ft. to an inch. No great accuracy is needed in this plotting, and it should be done by anyone not familiar with the use of different vertical and horizontal scales on the same drawing, a practice almost always used by surveyors and engineers laying out pipe lines.

SECTION II

Velocity Effects on Friction, Power, Etc.

Consideration has already been given to the power obtainable from water due to its elevation and to its pressure, but there yet remains one other source, namely, that due to its velocity or the power corresponding to the "kinetic energy." Most engineers are familiar with this, for they have often seen the jet from a fireman's nozzle and will realize that where a portable or mobile fire pump is used, all the power put into the water in the hose by the pump is either lost in friction or else comes out with the jet. The jet is at atmospheric pressure, and is usually held at the approximate level of the pump, so that there is no change in elevation and no gage pressure. For water turbines under high heads, the jets are often very powerful, being sometimes a foot or more in diameter and each delivering thousands of horsepower to the turbine runner.

Somewhere about three hundred years ago many studies were begun on the velocity of water coming from nozzles. Experiments previously made on solid bodies falling freely in air showed that the time required for the body to fall a certain distance did not depend on the weight of the body and that all bodies acquired the same velocity after falling the same vertical distance. For extremely light bodies like feathers, the relatively high air resistance affects this statement. The law for solid bodies falling freely

in air is that the velocity is proportional to the square root of the distance fallen and is expressed by the relation

$$v = 8.02\sqrt{h}$$

where v is the velocity in feet per second acquired after falling from rest through a vertical height h feet; the number 8.02 is the square root of 2×32.16 where 32.16 ft. per sec. for each elapsed second is the acceleration due to gravity. After a body has fallen 4 ft., its velocity from this formula ($8.02\sqrt{4}$) is 16.04 ft. per sec. and this is only doubled when it has fallen another 12 ft., or a total of 16 ft., because $8.02\sqrt{16} = 32.08$ ft. per sec.

Following these experiments, many measurements were made on jets. These showed that if a smooth and nearly frictionless nozzle was fixed into the side or bottom of an open tank containing water, then the velocity of water in the jet, when the surface of water was h ft. above it, was exactly the same as that of a solid body falling through the same height. Thus, if the depth of water above the nozzle was 4 ft., the velocity of the issuing water from the nozzle was found to be 16.04 ft. per sec. ($8.02\sqrt{4}$), and a similar law applies to all heads and to all nozzles, and likewise to smooth orifices in the side of the tank.

Relation Between Velocity and Power

This introduces an interesting and important idea, for it shows that, as far as power is concerned, it is possible to substitute an additional head to compensate for the velocity. If, at a point on a pipe, the pressure by the gage is 60 ft. and the velocity is 16.04 ft. per sec., then the power available at that point, due to these two, would be the same as that obtainable from water in an open tank where the surface of the water is $60 + 4 = 64$ ft. above a connected pipe. The 4 ft., or the height necessary to produce the 16.04 ft. per sec. velocity, is commonly called the "velocity head" and is given by $h = v^2/64.32$, which is only another form of the relation already given. On this basis the "total head" available for producing power is found by adding the height of the gage above the datum plane (elevation head), the pressure in feet indicated on the gage (pressure head), and the velocity head.

This statement is, in effect, that if at any point on a pipe line situated, say, 6 ft. above the selected datum, the pressure gage indicates 60 ft. of water, and the velocity of the water is known to be 16.04 ft. per sec., then the actual power in each pound of water flowing in one minute through the pipe is the same as if the pound of water was going over a waterfall $6 + 60 + 4 = 70$ ft. high, the values 6, 60 and 4 ft. corresponding to the elevation head, the pressure head, and the velocity head, respectively.

Friction in Pipes and Height of Fire Streams

Velocity head is of very great importance in hydraulic work, as subsequent examples will show. Friction losses in pipes, elbows, water meters, etc., are almost exactly proportional to the velocity head, an idea familiar to most practical water works men who are being constantly reminded that the frictional loss varies with the square of the velocity; if a pipe carries 1000 gallons of water per minute its loss of head (power) due to friction is only about one quarter what it would be if the pipe carried 2000 gallons per minute. In water works distribution piping velocities are often around 4 ft. per sec., for which the velocity head is $v^2/64.32 = 16/64.32 = 0.25$ ft., which is small compared with pressure heads, shown by gages on the pipes, as these often exceed 100 or 150 ft. But in suction pipes of centrifugal pumps, for instance, where the pressure is often below atmospheric, a velocity of 13 ft. per sec. would correspond to a velocity head of about 3.63 ft., which might be extremely serious in its effect on the pump, and it is never safe to assume this head is relatively negligible without first finding what its effect will be.

The following simple illustration of the meaning of velocity head may be helpful. If a shot of any size is dropped from a tower 100 ft. high, its velocity, when it strikes the ground will be $v = 8.02\sqrt{100} = 80.2$ ft. per sec., and if, on the other hand, a shot leaves a vertical air gun with a muzzle velocity of 80.2 ft. per sec. and moves through quiet air, it will rise to a height of 100 ft. above the muzzle. Not uncommonly, water leaves a fire nozzle at 80.2 ft. per sec., and if the nozzle points

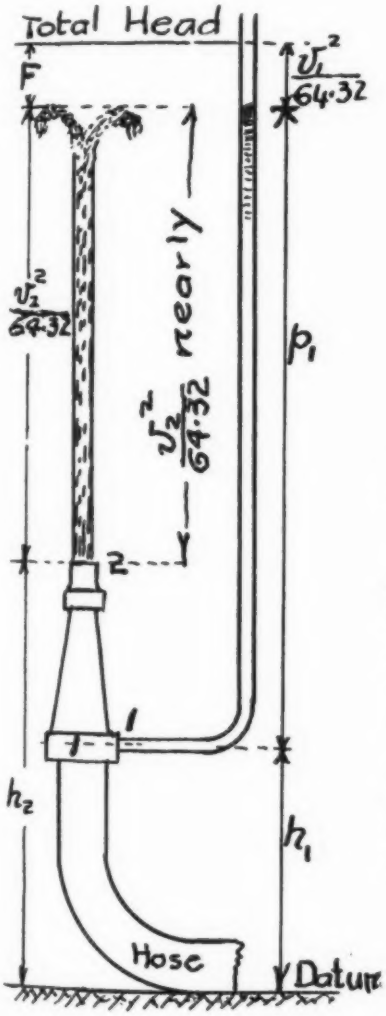


Fig. 11—Flow From a Fireman's Nozzle

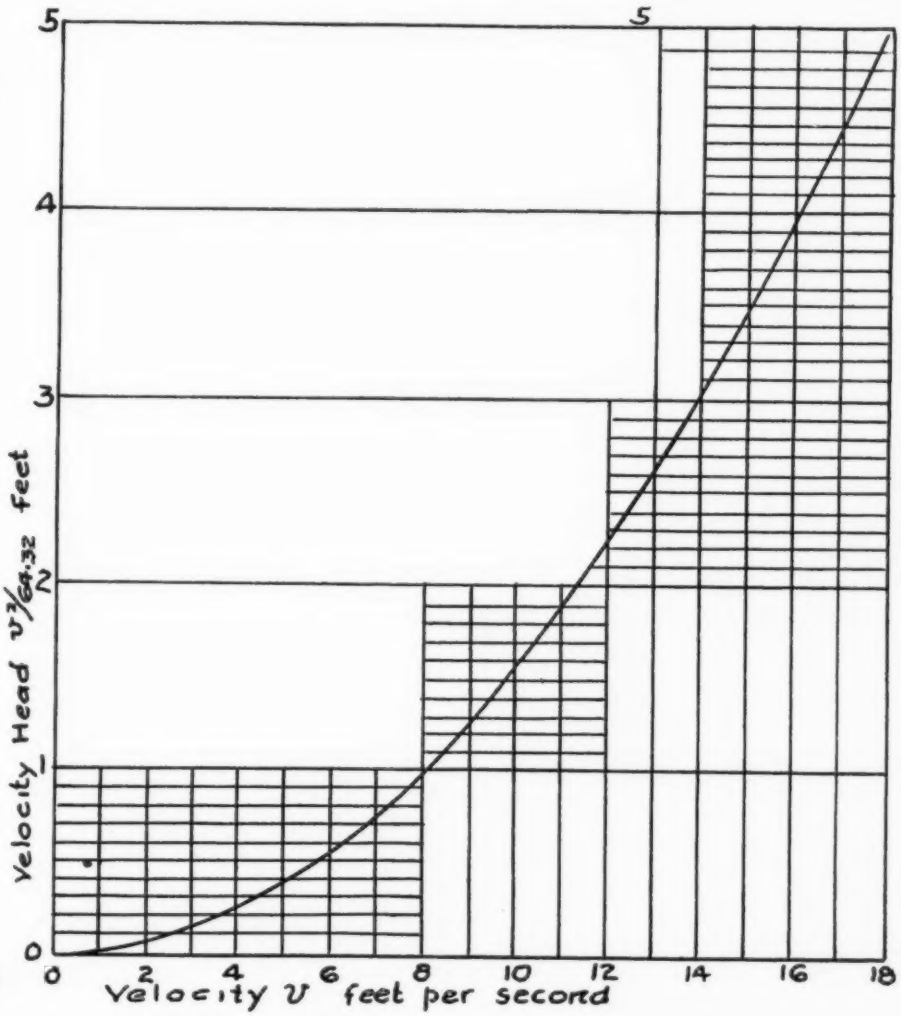


Fig. 12—Chart for Converting Velocities Into Velocity Heads

straight up and there is no wind, the stream should rise to a height given by $80.2 = 8.02\sqrt{h}$ or to a height h of 100 ft. Some of the highest drops actually do very nearly reach this height, but the air resistance has a tendency to break up the stream into spray and the air effect on small drops is very marked. Of course, this stream would not be used to fight a fire at anything near 100 ft. above the nozzle, and at heights considerably below this the energy in the water would be so low that the stream would be very ineffective. Actually the stream would not be used to reach a fire much over 60 ft. up.

This case is illustrated in Fig. 11, where the nozzle is pointing straight up and an open glass tube is shown attached at Section 1, at the base of the play pipe. The ground is taken as the datum and the elevation head is marked h_1 and the water rises in the glass tube by the pressure head p_1. The velocity head $v_1^2/64.32$ must also be added to give the total head at the base of the play pipe. Many tests have shown that the friction loss between the base of the play pipe and the tip of the nozzle is equal to the velocity head in the hose, that is, $F = v_1^2/64.32$.

At the nozzle the stream has a velocity v_2 and the jet would rise nearly $v_2^2/64.32$ ft. after leaving it, as already stated. For instance, if a pressure gage is attached at the base of the play pipe, as is often done by an inspector, and this gage reads 50 ft. pressure, then the highest drops of the stream will rise nearly 50 ft. above the gage.

For the convenience of those not familiar with the slide rule as a calculating instrument, in Fig. 12 is offered a curve of velocity heads corresponding to velocities up to 18 ft. per sec. In any problem the velocity in a pipe corresponding to a known discharge may be easily found from Table I and using this on Fig. 12, the corresponding velocity head *in feet* is read off.

SECTION III

The Pitot Tube and Fire Stream Gage

A very interesting application of velocity head is in the well-known Pitot tube used for measuring velocity of water and one very common application is in finding the discharge of fire streams, for which purpose inspectors find it most convenient. While somewhat elaborate forms of this tube are on sale and are most adaptable, any engineer can make one of his own, as the writer has done, by bending a short piece of small brass tubing and soldering to it a coupling so that an ordinary pressure gage graduated in feet may be attached to it. Such a simple form of the tube with its attached gage is shown in Fig. 13. It has already been stated that the power, per pound of water discharge per minute, due to its velocity v, is the same as it would be if the same pound per minute ran over a waterfall of a height of h ft., where the quantities are connected by the relation $v = 8.02\sqrt{h}$; or in another form, $h = v^2/64.3$. For example, this formula shows that where $h = 6.22$ ft., the velocity $v = 20$ ft. per sec. If a tube made as in Fig. 13 (bent so that the gage will be out of the way of the water jet when using the device) is held in a stream so that the open end faces squarely into the jet, then the gage should read 6.22 ft. if the velocity is 20 ft. per sec., because a column of water 6.22 ft. high could produce a velocity of 20 ft. per sec., and this would just balance the corresponding velocity

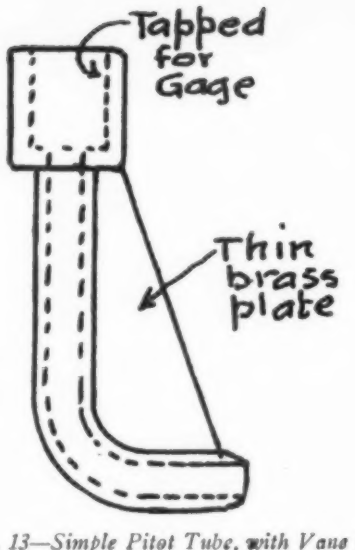


Fig. 13—Simple Pitot Tube, with Vane for Making Hydrant or Nozzle Flow Tests

energy in the jet. It is much the same as saying that if a piece of shot rolled off a shelf and fell down to the vertical muzzle of a tube 6.22 ft. below it, where it met another shot of exactly the same weight just leaving the muzzle at 20 ft. per sec., then the force produced by one of these pieces of shot would be exactly the same as that produced by the other and both would instantaneously come to rest.

The inspector inserts the tube into the stream in such a way that the bent part is along the center line of the stream, reads his gage pressure in feet, which is the velocity head h. He then calculates the velocity $v = 8.02\sqrt{h}$, and multiplies this velocity by the area of the nozzle to

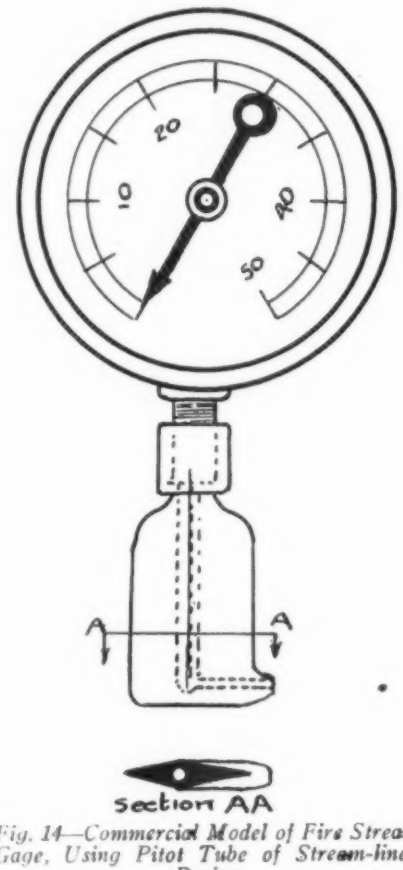


Fig. 14—Commercial Model of Fire Stream Gage, Using Pitot Tube of Stream-lined Design

get the discharge. For example, with a 1¼ in. parallel nozzle, which has an area .00852 sq. ft. (Table I), if the gage reads 35.8 ft., then the velocity will be $v = 8.02\sqrt{35.8} = 48$ ft. per sec., and the discharge would be $48 \times .00852 = .409$ cubic feet per second, which is easily shown to be 184 gallons per minute, since from Table I 450 gallons per minute is the same as one cubic foot per second. This tube may be discussed in greater detail later, but it should be stated here that the pressure in the fire stream as it leaves the nozzle is atmospheric, and also that a great many measurements made on fire streams show that at the nozzle all drops of water have almost identical velocities, so that the gage reads practically the same whether the tube is near the center or near the side of the jet. Convenient tables have been made up which give the discharge in gallons per minute for each gage reading and for several different sizes of nozzles, and these tables are issued free as an advertisement by some fire engine builders.* While the ring nozzle is not in very common use, it should be noted that it only gives about 74 per cent of the discharge of the conical nozzle of the same size and with the same gage pressure; the reason for this will be explained later.

While the simple form of Pitot tube is satisfactory for lower velocities, it is not structurally strong enough for the higher ones. The reason is that the tube must be kept of small size to prevent interference with the stream and it is therefore apt to vibrate when used on the larger nozzles and discharges. The instrument as sold for fire stream measurement is usually of the form shown as Fig. 14, the back of the gage sold with the instrument being graduated so that the discharge for any size nozzle and any gage reading may be read directly. Those using the device considerably will find the commercial models very handy, but if only a few measurements are to be taken, the home-made form is perfectly satisfactory and accurate.

The Pitot tube is also used to measure the velocity in a water pipe and is then modified in form because the pressure in the pipe must be taken into account: in case of the fire stream the pressure is atmospheric. In a water works main the pressure in the tube which faces upstream is due to the velocity plus the pressure in the main. Consequently, it is necessary to subtract the pressure in the pipe from the reading of the gage attached to the Pitot tube. This could, of course, be done by attaching a second pressure gage to the wall of the pipe so as to read the pressure, and by subtracting this reading from that of the gage on the Pitot tube the velocity head is readily found. Such a method is of little practical help because readings cannot be taken with sufficient accuracy as the following illustration shows. A common velocity in distribution mains is 4 ft. per sec., for which $v^2/64.3 = 0.25$ ft. (see Fig. 12), while pressures of (80 lbs. per sq. in. or) 185 ft. are not unusual, so that for this case the Pitot tube gage would read 185.25 ft. and the pressure gage would read 185 ft. but everyone knows that the gages could not be read accurately enough to be sure that the difference between them was 0.25 ft.

In measuring the velocity in a pipe under pressure, therefore, a special form of Pitot tube (such as the "Pitometer") is used

*Such a table is also to be found on page R-62 in the 1946 Reference and Data Number of *Water & Sewage Works*—April, 1946—under "Hydrant Discharge Measurements."—Ed.

and a special form of gage, referred to as a differential gage, is attached to it so that the differential gage automatically reads the velocity head only or, in other words, in the preceding example it would read the 0.25 ft. directly. If an ordinary pressure gage could be made with a watertight case which could stand high pressure, then if the gage was connected in the ordinary way to the Pitot tube and if at the same time a connection is made from the wall of the pipe to the watertight gage casing, then the spring tube in the gage would be subjected on the inside to the sum of the pressure and velocity heads, and on the outside to the pressure head only, so that the gage would indicate the velocity head only. Practical difficulties prevent the use of this construction and the differential gage is commonly a U-shaped glass tube containing mercury or air or some other fluid which separates the water columns produced by the Pitot tube and the pipe wall and gives the velocity head in very accurate values.

attached to this line, let us say 2000 ft. apart, their pressures must be connected by the relationship: elevation head + pressure head + velocity head at the upstream gage point must equal elevation head + pressure head + velocity head + friction head (lost between them) at the second section where the lower gage is.

Pressures in a Pipeline

To illustrate this important principle, suppose the line, Fig. 15, is made up of 1000 ft. of 10 in. pipe joined to 800 ft. of 8 in. pipe, and that there is a downward slope in this distance of 10 ft. This line is discharging 1200 gal. per min. and the gage at the beginning of the 10 in. pipe reads 115 ft. (50 lbs. per sq. in. approximately), what should be the reading on the lower gage (the one at the lower end of the 8 in. pipe) if the friction loss is 10.5 ft. up to the junction and 36 ft. in the entire line.

Table I, which is again reproduced here for convenience, shows that the velocity in the 10 in. pipe is 4.89 ft. per sec. and that in the 8 in. pipe is 7.65 ft. per sec., and for these velocities the curve in Fig. 12 (see section II) shows the velocity heads to be 0.37 ft. and 0.91 ft., respectively. (The unusually high velocity in the 8 in. pipe was chosen to accentuate the points being discussed.) The datum has been selected 20 ft. below the gage point on the 10 in. pipe, and as the 8 in. pipe is level, the elevation heads at sections 1, 3 and 2 are 20 ft., 10 ft., and 10 ft., respectively, as marked on the drawing.

Since the pressure head at section 1 is $p_1 = 115$ ft. and the velocity head there is 0.37 ft., the resulting total head is 20 + 115 + 0.37 = 135.37 ft. At section 2 the elevation head is 10 ft., the velocity head is 0.91 ft., and the loss between sections 1 and 2 is given as 36 ft., which means that the gage at 2 should read 135.37 — 10 — 0.91 — 36 = 88.46 ft., or 38.3 lbs. per sq. in. If the loss at the junction is neglected, then gages at **C** and **D** would have those readings shown on the drawing at these points. The results will, of course, be the same no matter where the datum is. The writer always finds it best to make a scale drawing of the pipe line and to plot the results as shown on Fig. 15. This can easily be done on a sheet of ruled paper with no other instruments than a divided scale and a pencil (and an eraser). If a vertical scale, such as 1/32 in. to 1 ft. and horizontal scale such as 1/32 in. to 10 ft. is used, an ordinary rule divided in 16ths of an inch is sufficient. The method of computing the friction loss will be discussed later, but in the meantime the reader should think out how resistance losses vary, and satisfy himself that the lines are in general of the correct shape and slope, for time spent in this way is far from wasted.

In the earlier discussions of pipe flow the effect of velocity head was not considered because it was very small compared with the pressure head and neglect of it at first enabled ideas to be introduced one at a time. The *hydraulic grade line* or *hydraulic gradient* was taken as the line which was always the friction head (**F**) below the line of total head. Actually the hydraulic gradient is below this by the amount of the velocity head in every case. One way to picture it is to imagine a series of open topped vertical glass tubes tapped into the pipe wall as in Fig. 15 and into which the water will rise to a height corresponding to the pressure. The hydraulic gradient is the line joining the tops of these water columns, but where there is a vacuum in the line, the outer end of the glass tube must be immersed in a pail of water and the hydraulic gradient falls as far below the center of the pipe as the water in the tube stands above the water in the pail. This is shown in the next illustration.

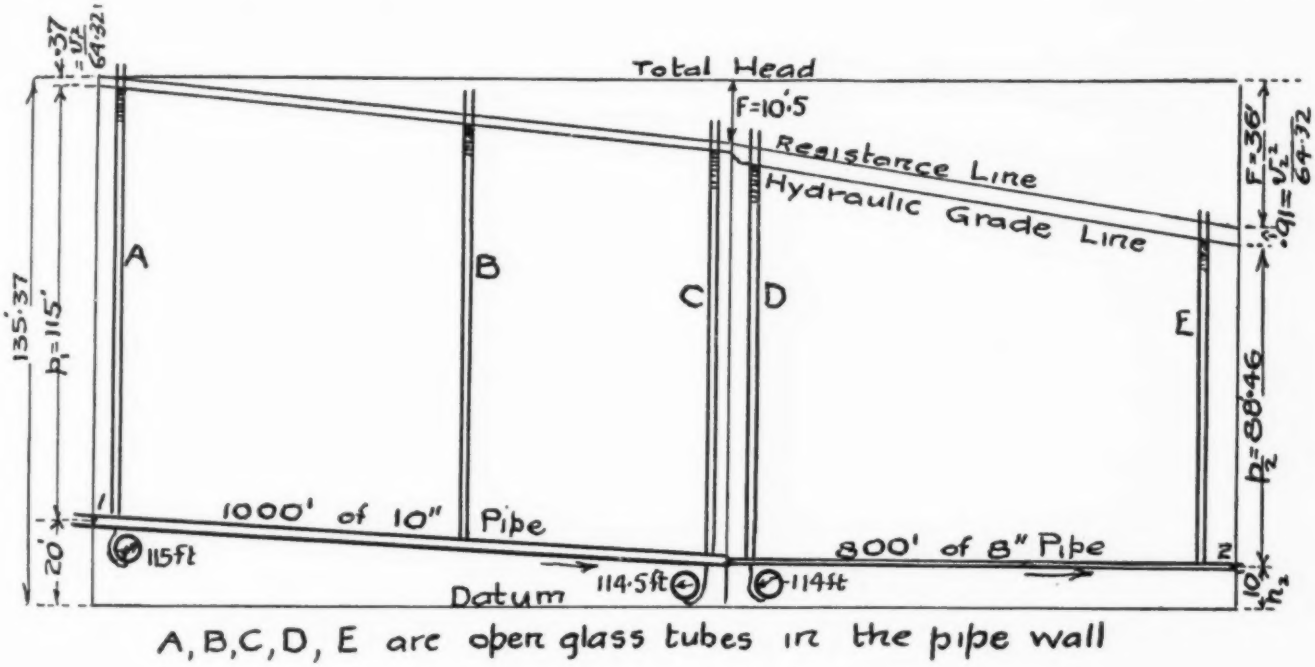


Fig. 15—Flow in a Main of Variable Size Pipes.

SECTION IV

Hydraulic Lines; The Venturi Meter; Measuring Flume; and Pump Suction and Discharge Pressures

Returning to the discussion of flow in pipes, it was stated in Part 2 of this series that if at a place on a pipe line situated 6 ft. above the selected datum there is a velocity of the water of 16.04 ft. per sec. (corresponding velocity head 4 ft.) and the pressure gage reads 60 ft., the total power in the water at this point will be the same as if the water was available at a waterfall 6 + 60 + 4 = 70 ft. high. This may be stated in the form: The power available in water flowing in a pipe, referred to any selected datum plane, is proportional to the sum of the *elevation head, the pressure head* and the *velocity head* at that point; consequently, if there is a pipe laid along a town or city street, where there are no branch connections and no pump or power consuming machine on the line, then if gages are

The Venturi Meter

As a further example we may examine the Venturi meter shown in Fig. 16. It is a horizontal 12 in. x 6 in. meter, discharging 2400 U. S. gal. per min. Table I gives the upstream and throat velocities as 6.8 and 27.2 ft. per sec., respectively, while Fig. 12 (see Section II) gives the corresponding velocity heads as 0.72 ft. and 11.50 ft. The friction loss in these meters is always small and will be taken as 0.8 ft. up to the throat, and by taking the datum along the center of the meter all the elevation heads automat-

ically become zero. Clearly, at the upstream end the total head is the sum of the pressure head and the velocity head, and if a gage attached there reads 10 ft., the total head will be $10 + 0.72 = 10.72$ ft. At the throat the pressure head plus 11.50 ft. plus 0.8 ft. friction loss must give 10.72 ft., or there is a negative pressure (i.e., a vacuum) of 1.58 ft. at the throat.

The various hydraulic lines for the meter are shown on Fig. 16 and are drawn for an overall loss in the meter of 1.8 ft. If the resistance lines are assumed straight as shown, it is a relatively easy and instructive task to plot the exact shape of the hydraulic grade line, because the diameter of the cone at each point is known and the velocity head at each point is readily found and plotted.

Another illustration of the effect of velocity head is shown between bridge piers where the water is lower than above and below them (where the piers themselves do not offer too much resistance) on account of the higher velocity between the piers. This fact has been employed in the design of certain measuring flumes such as the Parshall flume described on page 346 of the November, 1945, number of this magazine.

The Measuring Flume

The principle of the Measuring Flume is well explained by referring to experiments described in *The Engineer*, Aug. 3, 1934. These experiments were made in connection with an open box designed as a water meter. This consisted of an open horizontal box of the shape shown on Fig. 17, the dimensions and water depths reproduced corresponding to one of the tests. As the friction loss was so small that it affected the results less than half of one per cent, it will not be taken into account, and as the datum has been taken along the bottom of the box, all elevation heads are zero. At each section the velocity is found by dividing the discharge by the area of the section in square feet, and since the velocity squared divided by 64.32 gives the velocity head, it follows that at any section the velocity head is given by $Q^2/64.32\ A^2$, where Q is the discharge and A is the area. The areas are known and it is desired to find Q from the given measurements.

In an open trough such as this, the pressure head at any point is evidently just the depth of the water at that point, so that the hydraulic gradient is always at the water surface and the velocity head, in feet, plotted above this surface will locate the resistance line. Since friction is neglected, the latter is also the line of total head, and the change in depth between sections 1 and 2 represents the difference in velocity heads between these points. But this difference in depths, and therefore of velocity heads, has been given as $14.5 - 11.43 = 3.07$ in. or 0.256 ft.

The area A_1 is $13.3 \times 14.5/144$ or 1.34 sq. ft. and that at the narrow section is $4.91 \times 11.43/144$ or 0.39 sq. ft. which is A_2. The corresponding velocity heads are then $Q^2/64.32 \times 1.34^2$ and $Q^2/64.32 \times 0.39^2$, and when these results are worked out and subtracted they give $Q^2/10.67$. This is to be set equal to 0.256 ft. and from this the calculated value of the discharge Q is 1.653 cfs. Since the measured discharge in this case was 1.652 cfs., this method of measuring is very accurate, but in the practical use of the method some care is necessary and the reader would do well to refer to the original articles mentioned above. It is an excellent illustration of the meaning of velocity head and its application.

Suction Pipes on Pumps

Another interesting case which seems to give trouble to many water works men arises in connection with centrifugal pumps. Suppose that such a pump has a vertical suction pipe leading down into the well, the pump being placed 12 feet above the water and that a vacuum gage is attached to the pipe 8 feet above the surface of the well water. What should be the reading of this gage? Applying the principles already discussed and taking the datum at the surface of the water, the elevation head, pressure head and velocity head for a pound of water at the surface are each separately zero, so that the line of total head is at the water surface.

Where the gage is attached, the elevation head is 8 ft. and there is also the pressure head and the velocity head to be considered. There will also be some friction loss in the suction pipe up to this point, but the pipe is short, and if it has a bell-mouthed entrance without foot valve or strainer, the loss will be small. (With long, crooked pipes and sticky foot valves this suction pipe loss may be very serious.) This problem may best be studied by selecting an actual case of a pump for 2600 gal. per min. with a 10 in. suction pipe in which the velocity would be 10.6 ft. per sec. from Table I.

While it would be an improvement to put on an enlarger and increase the size of this pipe, it will still be helpful to examine the case as given, and it will be assumed that proper precautions have been taken in the design so as to keep the friction loss below the gage down to one foot. The curve on Fig. 12 shows that the velocity head for a velocity of 10.6 ft. per sec. is 1.75 ft. and, therefore, at the gage connection the elevation head of 8 ft. plus the pressure head (at present unknown), plus the velocity head of 1.75 ft., plus the friction loss of 1 ft. must equal the total head in one pound of water in the well, which is zero, as already explained. Therefore, $8 + p + 1.75 + 1 = 0$ or $p = -10.75$ ft., that is, there is a negative pressure or vacuum of 10.75 ft. at the gage. Had the pipe been enlarged to 12 in., the velocity would have been 7.37 ft. per sec. and the velocity head 0.84 ft., while the friction loss would have been about 0.5 ft., and the gage would have indicated a vacuum of 9.34 ft., found in the same way as before.

The enlargement of the suction pipe to 12 in. would mean a permanent reduction of 1.41 ft. head against which the pump had to work and would also improve the suc-

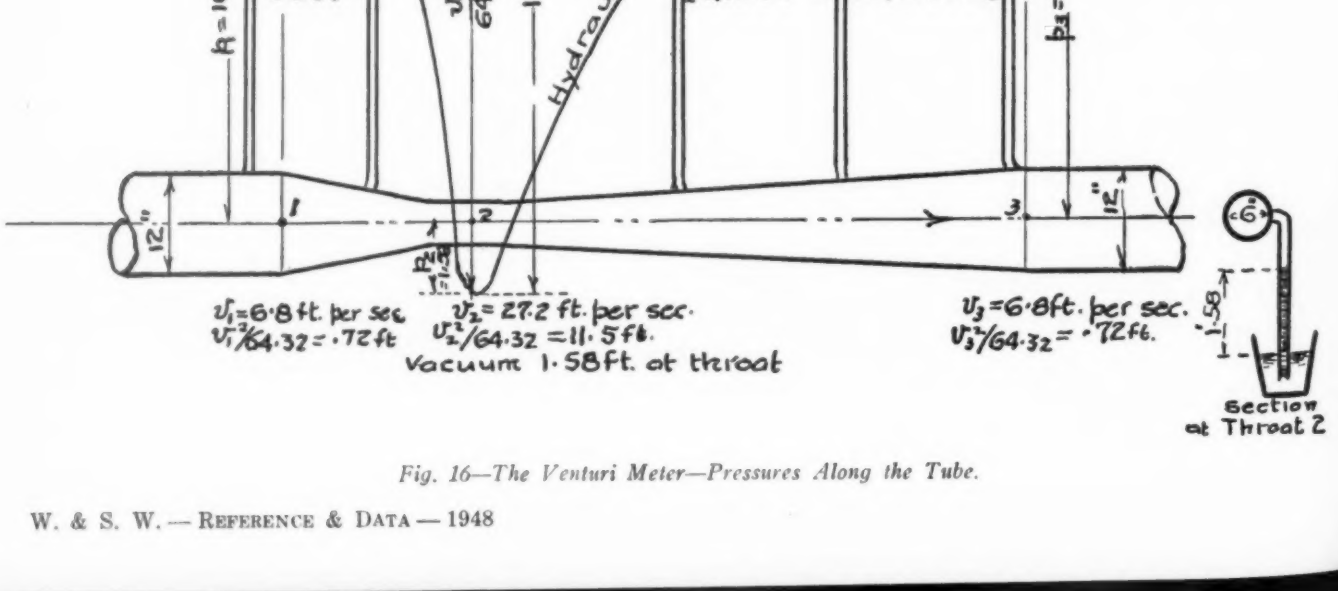


Fig. 16—The Venturi Meter—Pressures Along the Tube.

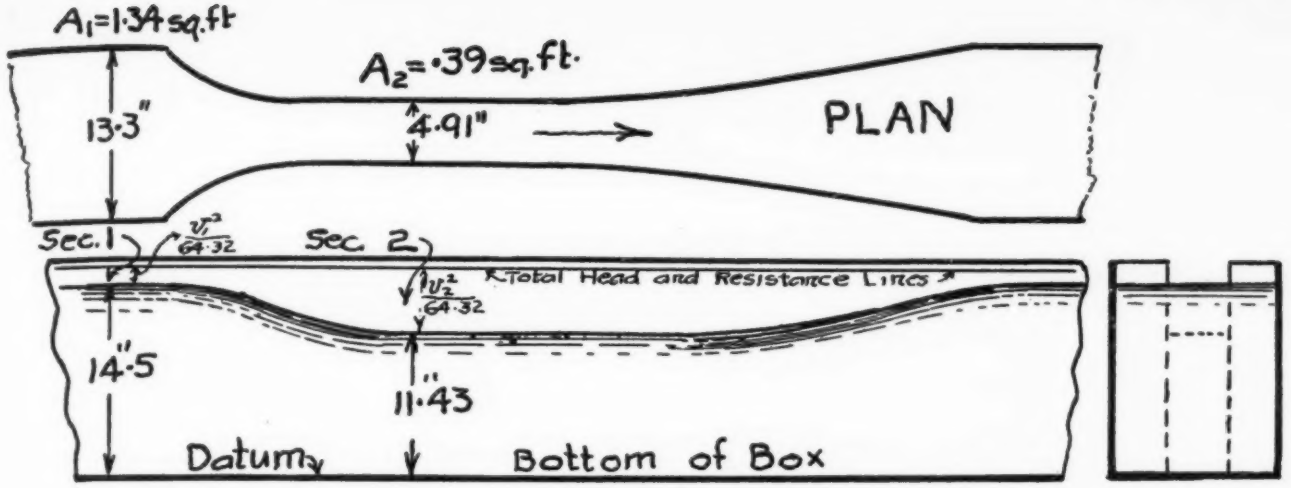


Fig. 17—The Open Measuring Flume.

tion conditions at the pump, so that the use of the larger pipe would usually be justified.

Before leaving this case, one other matter will be considered. The pump referred to had a very high efficiency, and in addition to the 10 in. suction branch it had an 8 in. discharge branch, in which the velocity was 16.56 ft. per sec., as calculated from Table I. The corresponding velocity head is 4.25 ft., from Fig. 12, and of the total head produced by the pump 4.25 — 1.75 = 2.5 ft. will be used up in creating this difference in velocity heads alone, and while there is no disadvantage in this so far as the pump is concerned, it may result in marked loss unless proper precaution is taken with the piping used.

To illustrate, suppose the pump lifts water from a well to a reservoir with its water surface 50 ft. above that in the well. Let the discharge piping be 8 in. and the suction piping 10 in., short and smooth and free from elbows and valves creating friction. Then the total friction losses in all the piping is about 13 ft., and the pump will have to create a net head of 63 ft. to overcome the friction and also the difference in elevation heads. Now suppose the 8 in. discharge pipe projects horizontally through the wall of the reservoir, the water entering the latter at 16.56 ft. per sec., and that the center line of the pipe is 4 ft. below the water surface. Before pumping begins, a gage on this pipe close to the reservoir will read 4 ft. pressure, and numerous experiments show that this reading will be practically unchanged at full discharge, which means that there will be a loss of head of 4.25 ft. (the velocity head) at exit from the pipe.

To prove this, the datum will be taken through the center line of the pipe and, therefore, the sum of the pressure head of 4 ft. and of the velocity head of 4.25 ft. at the gage point must equal the sum of the pressure head and the velocity head for a pound of water in the reservoir, plus the friction or other loss between the gage and the reservoir. In the reservoir at the datum level the pressure head is 4 ft. and the velocity head is zero, since the water is not flowing there, so that the total head of 8.25 ft. at the gage point equals the 4 ft. in the reservoir plus the losses between them. Thus the losses where the water leaves the pipe will be 4.25 ft., the velocity head in the pipe.

This means, of course, that the pump has to lift the water 50 ft. to overcome the difference in levels, plus 13 + 4.25 = 17.25 ft., to overcome the friction and discharge end losses, and the total head to be pumped against is, therefore, 67.25 ft. If it is assumed that ten of the thirteen feet friction loss is in the discharge piping, then it is easy to show that the discharge gage at the pump should read 44 ft. if the gage is located 16 ft. above the well.

Obviously the losses are out of all reasonable values compared with the net lift of 50 ft. obtained and it would be most desirable to increase the sizes of the pipes. Suppose that enlargers are put on the pump flanges so that the suction and discharge lines may be increased to 12 in. and 10 in., respectively, then the velocity head in the suction pipe will fall to 0.84 ft. and that in the discharge pipe to 1.75 ft. Further, the friction losses will be about one-third their former value and may be taken as 4.3 ft.

Since the discharge end loss, corresponding to the velocity head in the discharge pipe, is now only 1.75 ft., the total losses are 6.05 ft. instead of the 17.25 ft. in the former case, and the total head to be produced by the pump is 56.05 ft. to give the net lift of 50 ft., or a saving of 11.20 ft. of head. With the larger piping the power required for pumping would be 56.05/67.25 or 83.3 per cent of that required if the smaller pipes were used, and this saving of 16.7 per cent can generally be made at relatively low cost. While the high velocities often used in the pump nozzles are not objectionable as far as the pump itself is concerned, these high velocities should always be avoided in piping, particularly on the suction side of the pump.

The method of calculating the friction losses, knowing the velocities and pipe sizes, will be taken up in the next section, but the values used in this example are reasonable. Reduction of friction losses and those corresponding to velocity heads is one of the main problems of the engineer.

SECTION V

Friction and Resistance Losses; Pipe Line Losses; Roughness Effects

In earlier articles of this series reference has frequently been made to friction and resistance losses. This matter will now be examined in some detail, because friction losses in piping and other channels in which liquids flow is a most important matter. Dealing first with pipes, it is well known that the loss of pressure due to friction depends on the length of the pipe, the size of it, the velocity of the flowing water and on the condition of its internal surface. For pipes that are equally smooth inside, the friction head lost in pipes of the same size increases with the length of the pipe, as examples already given in previous articles indicate. Now, in addition, head loss also increases with the discharge increase through any definite pipe. In fact, many experiments show that for a particular pipe the loss of head due to friction varies almost directly as the square of the velocity of flow through the pipe. More generally stated, the friction loss varies directly with the velocity head.

Roughness of Pipes

The factor of greatest importance in connection with head loss due to friction is the condition of the internal surface of the pipe, more generally described as the *roughness* of the pipe. This is also the most elusive of all factors entering this problem, for no one has yet discovered an accurate means of classifying roughnesses, and the experimental measurement of the friction losses is very often the only way of comparing the roughness of pipes. It is true that the hand and eye can be of great assistance, for where pipes feel rough they are rough so far as the flow is concerned, and where lumps or projecting pieces of metal are observed the same is true.

On the other hand, a pipe may feel smooth to the hand and may yet offer much resistance to flow because it is not round, or because it has slight corrugations in it. A smooth pipe must not only feel smooth but it must be uniform in size, truly cylindrical, and free from bends. Good examples are drawn copper, lead and brass pipes, but iron pipes are often also very smooth, especially after they are coated or cement lined. Cement-asbestos and concrete pipes are often very superior in this respect, but even in the smoothest and best pipes the friction losses are not negligible, for the water flowing over the stationary pipe wall suffers considerable resistance.

In the game known as curling, the stone moves over ice, which is of course very smooth, yet there is enough friction to eventually stop the stone. If the ice has lumps on it, the stone stops sooner than otherwise, even though the lumps are smooth. Wavy ice produces much the same effect as ice with lumps and there is usually no way of predicting the effect of these factors except by making a few trials, and this is only done on ice which has been made as smooth as possible.

Friction Head and Power Lost in Friction

The friction loss or friction head is measured in feet or in pounds pressure per square inch and has been so indicated in the examples given. Since this leads to some confusion, an illustration or two may prove helpful. A car not having too much friction in its parts will, of itself, roll down a sloping road and will run at uniform speed if the slope is small, in which case the working of gravity along the slope supplies just enough power to overcome the friction of the car on the road. It gets its power in this case by "losing altitude," and if this loss of altitude is 10 ft. while the car moves down the slope a distance requiring 25 sec., then the power put into the car by gravity is its weight multiplied by the 10 ft. and divided by the 25 sec. required. This gives the power in foot pounds per second, and by dividing this power by 550, which is the number of foot pounds per second corresponding to one horsepower (33,000 foot pounds per minute), the horsepower lost in friction is found, assuming the car moved at uniform speed. If the above car weighed 4000 pounds, the power lost in friction was 2.91 hp., but the "head" lost was 10 feet, regardless of the weight of the car or any other factor.

Such a loss in altitude (pressure) is, in hydraulics, referred to as the friction head and is clearly independent of the weight moving or the time taken; it is simply a number of feet.

In dealing with hydraulic problems, the friction head loss, in feet, is first calculated, and from this the horsepower lost in friction is found by multiplying this head loss by the *weight* (rather than volume) of water flowing per second and dividing this by 550. If, for example, the friction head loss is 6 ft. in a pipe carrying 4 cu. ft. per sec., the power lost in friction is

$$\frac{4 \times 62.4 \times 6}{550} = 2.72 \text{ hp.}$$

(1 cu. ft. of water weighs 62.4 lbs.)

It is important at the outset to get the distinction between the head lost in friction and the power lost in friction. The former is a number of feet and is the quantity plotted on the diagrams accompanying previous articles.

Friction losses are so serious in hydraulic work that much experimental work has been and is being done in order to formulate laws for friction loss calculations. For pipes, the number of recorded experiments is nearly infinite and these form the main basis of our knowledge, although theory has also been helpful. That the friction head *increases* at the same rate as the pipe length increases seems to be obvious, but it is not quite so clear why friction should *decrease* so markedly as the diameter of pipe increases, although a little consideration will show this to be the case.

Effect of Size on Friction Loss

Since the cause of all the friction loss is obviously the drag produced by the stationary pipe wall on the moving water, one may reason that the further the average particle of moving water is from the wall the less should be the friction. In a 6 in. pipe no particle of water can be more than 3 in. from the wall and the friction effect must be greater than in a 12 in. pipe where some of the particles are 6 in. away from the wall; in fact, it must be much less in the latter case as the average drop of water is twice as far away from the wall as it is in the small pipe. Whether or not one would expect the effect to be as great as it is, experiments show that in such a case as that given, the friction head loss in the large pipe would be just almost exactly half what it would be in the smaller pipe, provided both were equally smooth and both carried water at the same velocity.

Care must be used in this matter, however, for the larger pipe delivers four times the water that the small one does, because its area is four times as large while the velocity is the same in both. Therefore, while the *loss of head* is only half as great in the large pipe, the actual horsepower lost in the large pipe is twice that in the small one. Therefore, it is the loss of head that decreases at the same rate as the diameter increases.

Formula for Friction Head

Experiments also show that the loss of head in a particular pipe varies almost exactly in proportion to the velocity head, as has already been stated, and when the effects of length, velocity and diameter are combined, the loss of head is expressed by the formula

$$F = f\frac{l}{d} \times \frac{V^2}{64.3}$$

where the length **(l)** and diameter **(d)** are in feet and **(f)** is the *friction factor,* and depends almost exclusively on the roughness of the inside of the pipe. Curiously enough, experiments show that there is actually a small variation in **(f)** as the diameter and the velocity vary, but the latter have not a very great effect in pipes of uniform roughness and it only means that the formula does not take exact account of the effect of diameter and velocity. But it is practically very much easier to calculate the friction loss by this simple law, with an accompanying table of suitable friction factors, than it would be by the use of a much more complicated formula which would take exact account of size and velocity in such a way as to make **(f)** depend only on the roughness.

Friction Factors

The friction factor table in common use is a very old one and was made for straight, circular pipes of about the same internal roughness as is found in tar coated cast iron water pipes commonly used for water works service. The factors are too high for such smooth pipes as brass and copper and are too low for rust encrusted cast iron pipes, but a few values as given in Table II will be found useful in most cases occurring in practice, and will also serve to show how the factor is affected by size and the velocity of flow.

As an illustration, a nearly new 6 in. cast iron pipe is delivering 500 U. S. gal. of water per min.; what would be the friction loss in a length of 2000 ft.? From Table I the velocity is found to be 5.66 ft. per sec. From Fig. 12 the velocity head is 0.50 ft., while Table II gives the friction factor as 0.0235 approximately, so that the loss of head is

$$F = .0235 \times \frac{2{,}000}{6/12} \times .50 = 47 \text{ ft.}$$

If it was desired to increase the discharge 50 per cent, i.e., to deliver 750 gallons per min., then the velocity would be 8.49 ft. per sec., the corresponding velocity head 1.12 ft., and the friction factor would be 0.0226 (Table II), and if these quantities are put in the formula the friction loss is found to be 101.2 ft., or 2.15 times what it is for the smaller flow.

While on this matter it might be well to call attention to the effect of the size of the pipe on the friction loss. To somewhat exaggerate the case, let us suppose that a 12 in. pipe is used instead of the 6 in. pipe, and that it delivers the larger quantity of 750 gallons per min., which will mean the low velocity of 2.12 ft. per sec. This corresponds to a velocity head of 0.07 ft. and a friction factor of 0.0229 and the calculated loss of head **(F)** is easily found to be 3.21 ft. as compared with 101.2 ft. for the smaller pipe. It is scarcely justifiable to use such a large pipe for this job, as the velocity is very low, but if anticipated increase of demand in future as well as deterioration through age are considered, the larger pipe may be desirable. The calculation shows very clearly the effect of increasing the size of the pipe.

For convenience in solving this and similar problems, Fig. 18 has been prepared. It gives the loss of head in 1000 ft. of clean, new cast iron pipes of various sizes of from 2 in. to 30 in. and for the usual range of velocities. Separated from the other curves is one showing the loss in 100 ft. of 2½ in. fire hose for a good range of velocities, and it is to be observed that velocities in fire hose are much higher than in regular iron pipes used in water works service. As already pointed out, these losses are 5 per cent higher in riveted steel pipes and 50 per cent higher in many old and rusted pipes, so that some judgment must be used where the roughness is different from that used in plotting the curves. Of course, brass, lead and copper pipes have smaller losses than the diagram shows.

TABLE II
Friction Factors for Smooth, Clean, Coated Cast Iron Pipes
(Add 5% for riveted steel and at least 50% for old rusted pipes. Copper and brass and lead pipes have lower values.)

Pipe Diameter In.	Velocity in Pipe, Ft. per Sec.					
	1.0	2.0	3.0	4.0	5.0	10.0
1	.035	.032	.030	.029	.027	.024
2	.033	.030	.028	.027	.026	.024
4	.031	.028	.026	.025	.025	.023
6	.029	.026	.025	.024	.024	.022
12	.025	.023	.022	.022	.021	.020
16	.023	.022	.021	.021	.020	.019

TABLE III
Loss of Head in Fittings and Special Castings
(Expressed as the equivalent feet of straight pipe having the same loss.)

Screwed tee, or square elbow	70 ft.
Screwed short radius elbow	30 ft.
Screwed long radius elbow	20 ft.
Cast bell and spigot or flanged, long radius 90 deg. bend	10 ft.
Entrance loss where pipe projects into reservoir	30 ft.
Entrance loss where end of pipe is flush with inner wall	20 ft.
Entrance loss, entrance well rounded (bell month)	0 ft.
[Discharge end loss, where pipe is kept full diameter, discharge end submerged, is equal the velocity head in the pipe, or approximately 40 ft. of pipe.]	

Loss in Fittings, Specials and Valves

The discussion so far has been confined to round, straight pipes, but actual lines usually have elbows, bends, valves and other sources of loss which must be included. The loss of head in each of these cases is also proportional to the velocity head in the pipe and probably the most accurate way to deal with such losses is to multiply the velocity head by a *coefficient of resistance* found by experiment for each type of fitting or valve and many tables of these coefficients have been made. Some of these values are given in Table III. For the degree of accuracy demanded in ordinary work, however, the loss of head in a fitting or special is often expressed as the number of feet of straight pipe having the same loss, and this length is added to the length of straight pipe involved.

For example, the loss of head in a 90 deg. short radius 6 in. screwed elbow is 0.75 of the velocity head in the pipe. But Table II gives the average friction factor for 6 in. pipe as about 0.025, which means that a rough average value of the loss in 6 in. straight, smooth pipe is $0.025 \times 1/d$ times the velocity head. If 1/d is 30, this

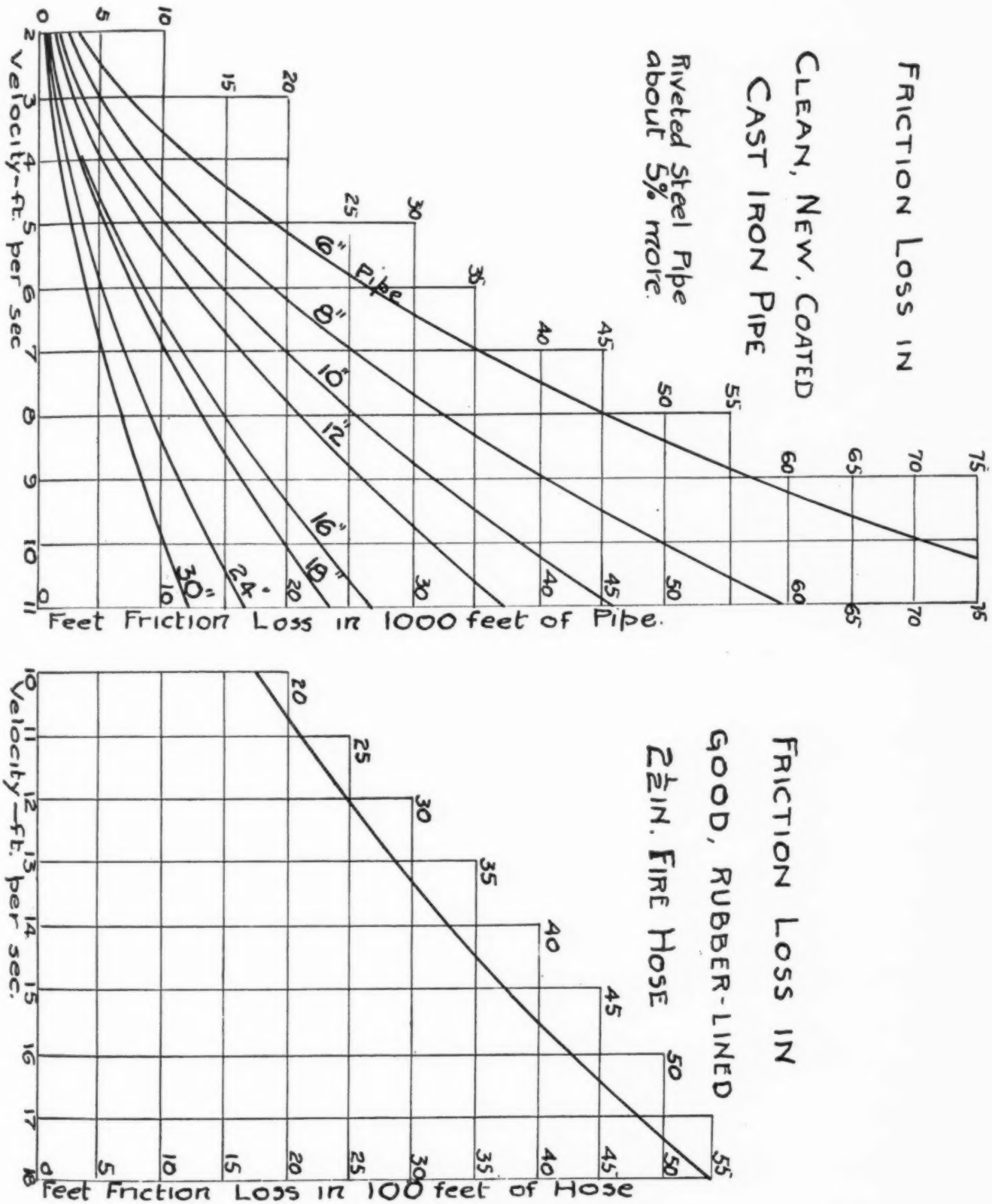


Fig. 18—Friction Losses in Pipes and Hose Lines

Fig. 19—Intake Loss and Pump Pressures

is equivalent to a straight piece of 6 in. pipe 30 × 6 = 180 in. long (i.e., 15 ft.) the loss of head in this piece would be 0.75 times the velocity head and it would be correct to say that if a 6 in. pipe had screwed 90 deg. elbows of short radius in it the loss of head in the line could be calculated as for straight pipe if for each such elbow the 15 ft. equivalent of straight pipe are added. Thus, 100 ft. of pipe with two of these elbows in it would have the same loss as 130 ft. of straight pipe, and so on.

The values given in Table III are typical, but it must be remembered that there are many types of elbows, bends, etc., and judgment is necessary in using the table. For example, in the case of the 6 in. elbow just mentioned the losses in the elbow will be much increased if burrs are left on the inside of the pipe ends and the tabular values are only to be used in well connected fittings. Since in most pipe lines the losses in fittings are usually relatively small compared with those in the straight part of the line, no serious trouble will arise if the values used for the fittings are somewhat inaccurate, but in many cases the loss in the fittings may easily exceed those in the straight part of the pipe and then care must be used in selecting proper values.

In the case of valves the losses are much more difficult to evaluate, because valves differ a great deal in their construction, and it is usually difficult to tell just how far the valve is open. In this case it is best to express the loss as a coefficient to be multiplied into the velocity head and Table IV gives a few such coefficients.

TABLE IV

Losses of Head in Gate Valves

(Expressed as coefficients to be multiplied into the velocity head in the pipe.)

Gate Valve Size, In.	Ratio of number of turns valve is open to number required for full opening			
	Full Open	¾ Open	½ Open	¼ Open
2	.16	.68	3.0	20
6	.12	.49	2.5	15
10	.07	.41	2.3	13

For Globe Valves, full open, the losses are much higher than for gate valves and the coefficient varies from around 14 on half inch sizes to around 6 on two inch valves, and will somewhat decrease with the larger sizes.

Just in passing, however, attention might be called to some striking results in Table IV, the first being the rapid way the losses rise as the valve approaches the closed position. As the valve closes it reduces the flow partly because the area through it becomes smaller but also because of the rapid increase in resistance in it; the combination of these two produces the final result. Certain modern valves have much lower friction losses than gate valves have, in which case their effect is largely due to the reduction of area in them. Another thing Table IV shows is that the loss is much lower for large valves than for small ones, but that the losses are not negligible even in the fully open valve.

Most of this section has had to be taken up with explanation of how to calculate friction losses, together with necessary data. The next section will apply the method here explained to some common practical problems.

SECTION VI

Intakes and Pressure Losses; Effects of Age on Pipes

In the previous section the method of calculating friction losses in pipe lines was explained and the necessary data given. To illustrate the application of the method, suppose that an intake pipe (Fig. 19) 2000 ft. long is laid from a pump well out into a lake and that it is desired to have the water in the well not over four feet below lake level under a demand of 2,400,000 U. S. gallons per day. The pipe is to have a smooth interior finish similar to that of coated cast iron pipe, and is to have a bell mouth entrance in which the loss is negligible, and further, there are no bends except the one shown at the end. What size pipe should be used?

The four feet difference in levels is simply the friction loss in the pipe (see Figs. 5 and 9) and a study of Fig. 18 would make it appear that a 16 in. pipe might be examined as a first trial. If this is the correct size, the velocity in it will be 2.66 ft. per sec. (from Table I) and the corresponding velocity head will be 0.11 ft. from Fig. 12. Further, since the entrance loss is negligible, the only losses will be those due to the straight pipe and the elbow, to which must be added, however, the exit loss at the well end. The friction factor is 0:0211 (Table II) and the elbow causes the same loss as 10 ft. of pipe, while the loss at the well end is the same as that in 40 ft. of pipe. The total loss will therefore be

$$F = .0211 \times \frac{2050}{16/12} \times .11 = 3.6 \text{ ft.}$$

which indicates that 16 in. pipe would be the right size.

Effect of Age on Pipe

It must be kept in mind that as the pipe gets older the friction rises, and in unfavorable cases may easily increase 50 per cent, which would bring the loss up to 5.4 ft., and if such a drop in well level is objectionable it would be better to install a larger pipe to begin with. Loss of head in an intake is usually not very serious from the power standpoint, but it produces a higher suction lift on the pump, which is often objectionable, especially if there is danger of the lake level lowering with time. If instead of the 16 in. pipe one 18 in. is used, the loss of head in the pipe when old would only be 3.1 ft.

Pressure Required for Pumping

Now suppose a single pump is used on this job and that it has a 10 in. suction pipe arranged as shown on Fig. 19, what would be the reading on the suction gage, corrected to the center line of the pipe? It is assumed that there is no foot valve or strainer, priming being separately arranged. For the 10 in. pipe delivering 2,400,000 gallons per day the velocity is 6.8 ft. per sec., for which Fig. 12 gives the velocity head as 0.72 ft., while Table II gives the friction factor as 0.22. If the length of straight pipe is 30 ft. and 10 ft. are added for the elbow (Table III) the equivalent length of straight pipe is 40 ft., in which the friction loss is

$$F = .022 \times \frac{40}{10/12} \times .72 = 0.76 \text{ ft.}$$

or practically 0.8 ft. (This may be checked on Fig. 18.) Applying the method described in the third article of this series shows that the gage will indicate a vacuum of 3.6 + 6.0 + .72 + .80 or 11.1 ft., which is not too high for a centrifugal pump.

In the last example consideration should be given to the fact that when the suction pipe is old and the intake also, this lift will be increased to around 13.3 ft., which is fairly high and will prove troublesome if increased by a drop in lake level. The pump should be set as low as possible, preferably below lake level, which offers the additional advantage of freedom from the necessity of priming.

Discharge Pressure at the Pump

Now let us suppose that the pump is delivering its full capacity through 4,000 ft. of 12 in. pipe that has been in service for about ten years and is a little corroded but is still in fair condition. There are eight right-angle bends and four full open gate valves in the line and it delivers water to a reservoir 180 ft. above the lake. What pressure would the gage on the discharge pipe, near the pump, show?

For this case the velocity in the pipe is 4.72 ft. per sec. (Table I), and the velocity head is 0.35 ft. from Fig. 12. Further, the friction factor of 0.0213 (given in Table II), must be increased by about 25 per cent on account of the old pipe and then becomes 0.0266, and since there are eight bends, each of which has the same loss as 10 ft. of pipe, the equivalent length of straight pipe is 4,080 ft. The four gate valves each give a loss equal to 0.06 of the velocity head, making a total for these of 4 × .06 × .35, or approximately 0.1 ft. The loss of head in this line is then

$$.0266 \times \frac{4080}{12/12} \times .35 + .10 = 38.1 \text{ ft.}$$

(This could have been taken from Fig. 18 by adding 25 per cent to the values given there.) We must finally add the velocity head 0.35 ft. (Table III) to allow for exit loss, bringing the loss to 38.5 ft.

Proceeding by the method described in Section III, which says that the sum of the elevation head, the pressure head and the velocity head at the gage must equal the elevation head plus the pressure head of a

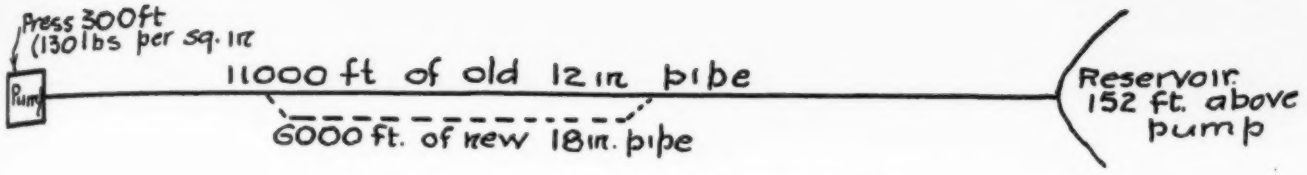


Fig. 21—Branched Pipe and Pump Pressure of 300 ft.

pound of water in the reservoir (where the velocity is zero), plus the loss of head in friction between the gage and the reservoir, we have the following relation, elevations being based on the lake level as datum: $6 + p + .35 = 180 + 38.5$, or the pressure p on the gage is 212.1 ft. The power delivered by the pump to the water is, of course, due to the difference in pressures of the suction and discharge gages (since both are at the same level), plus the difference between the suction and discharge velocity heads. The difference in pressures (added since the suction gage has a negative or vacuum reading) is $212.1 + 11.1 = 223.2$ ft., while the difference of velocity heads is $0.35 - 0.72$ or minus 0.37 ft., making a total head of 222.8 ft. to be pumped against. Note that the velocity head difference is a negative one since that in the suction pipe is the greater of the two.

The pump must, therefore, deliver 2,400,000 gal. per day, or 1,667 gal. per min., against 222.8 ft. head, and since a gallon weighs 8.33 lb., this requires a water horsepower of $1{,}667 \times 8.33 \times 222.8/33{,}000 = 93.8$ hp. If the pump is of high grade, it should have an efficiency of at least 80 per cent and the motor would need to be $93.8/0.8 = 118$ hp., the nearest commercial size to which might be 125 hp.

Effect of Roughening of Inside of Pipe Due to Age

Here again deterioration of the inside condition of the pipe must not be forgotten, for it means a decrease in carrying capacity for a given pump pressure. But all centrifugal pumps, when working near their maximum efficiency point, show a decrease in discharge as the pressure rises, and it will be of interest to examine the matter further. An interesting study of this problem is illustrated in Fig. 20, where a typical head-discharge curve (1) for a centrifugal pump is shown, the pressures plotted being those on the discharge gage. The pressure necessary to force different quantities of water through the discharge pipe is shown on the curves (2), (3) and (4), these latter curves being found by calculating, in the manner already described, the pressures for a number of discharges and in each case for the estimated friction factor corresponding to the age of the pipe. Curve (2) is for new pipe, while (3) and (4) correspond to increases of 25 per cent and 50 per cent, respectively, in the friction. These curves may, of course, be plotted by using Fig. 18 of the previous article.

In the diagram the pump is assumed to give its rated capacity of 2,400,000 gals. per day where the pipe is as described above, and Fig. 20 shows that the discharge will fall to 2,250,000 gal. per day for 50 per cent increase in friction above new pipe and that it was 2,550,000 gal. per day when the pipe was new. Unless provision is made to counteract it this loss may be serious, because the demand usually *increases* as time goes on.

Branched Pipes

In cases like that just discussed relief is often sought by putting in a new pipe and this problem will now be considered. As the first case, let us suppose that on a line of old 10 in. horizontal pipe two gages attached at points 2,000 ft. apart show a pressure drop of 20 ft., and if the pipe had been new, this would correspond to a discharge of 1,210 gal. per min., as is easily calculated or may be read from Fig. 18. But when the pipe is old and rusted, the friction factor might increase 50 per cent in unfavorable cases, and if it did, the discharge would fall to 973 gal. per min. for the same pressure drop of 20 ft.

In order to compensate for this decreased discharge without disturbing the gage pressure difference of 20 ft., let us suppose that

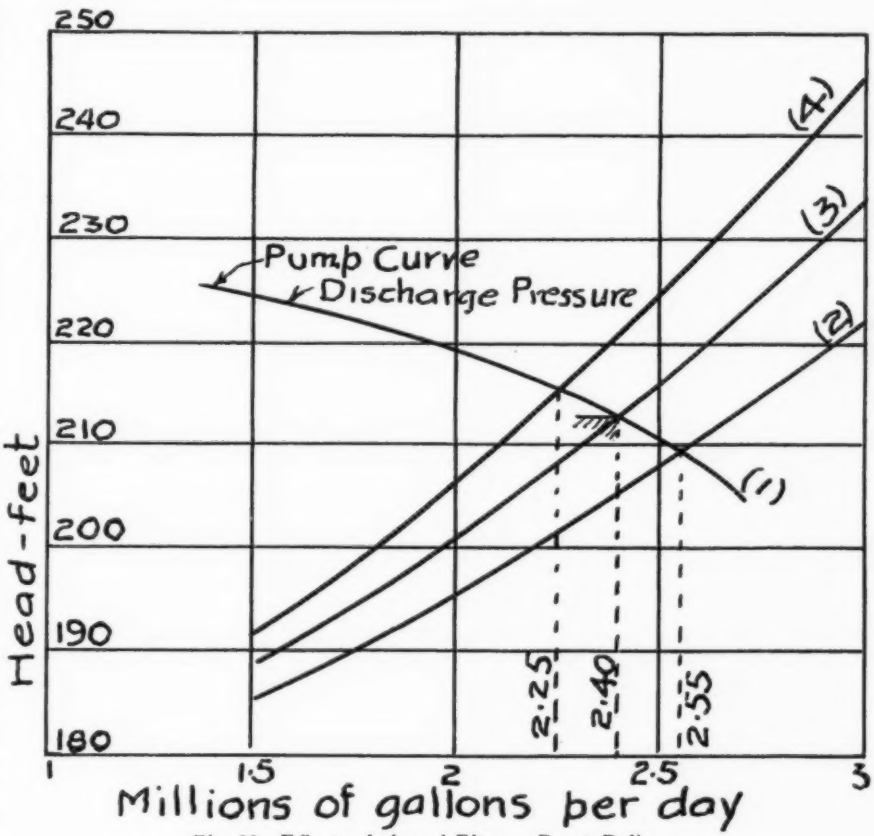


Fig. 20—Effects of Age of Pipe on Pump Delivery
(Curve 2 applies to new pipe; curve 3 to pipes where friction has increased 25%; curve 4 to pipes where friction has increased 50% due to age.)

a piece of new 12 in. pipe is laid in parallel with the 2,000 ft. of old pipe and that the new pipe is joined at both ends to the old one by Y branches and 45 deg. bends, which offer very little hydraulic resistance. What will be the flow for both pipes if the gage readings are unchanged? The first impression is that the water will divide between the two pipes in proportion to their areas, but this is wrong, and a little consideration will show that the division of flow is such that the loss of head in each of the pipes will be 20 ft. By use of Fig. 18 it is shown that this will correspond to a velocity of 5.55 ft. per sec. in the new piece, so that this pipe will carry 1,960 gal. per min. For the old piece the discharge will continue at 973 gal. per min., so that the two combined will deliver 2,933 gal. per min. between the two points, which is more than double what it was before.

Effect of a Branch on a Long Pipe

As a useful practical application let us examine the case illustrated in Fig. 21, where a pump has been discharging through 11,000 ft. of old 12 in. pipe to a reservoir, and the flow has diminished so much that it must be augmented. The reservoir is 152 ft. above the pump, and we shall suppose that the pressure at the pump can be maintained at 300 ft. or 130 lbs. per sq. in., so that the friction loss in the line is $300 - 152 = 148$ ft. This loss of head corresponds to a velocity flow of 6.5 ft. per sec. (see Fig. 18) or to 2,300 gal. per min. for new pipe of these dimensions. If desired this may be checked by the formula

$$F = f \frac{l}{d} \times \frac{V^2}{64.3}$$

in which F is 148 ft., l is 11,000 ft., f is 0.0206 (Table II), and d is one foot, and from these the velocity will be found to be 6.5 ft. per sec. as before.

Increased friction as the pipe becomes older will decrease this flow to a point where we shall assume it must be improved. The pipe might be removed and replaced by a new one, but this is rarely done, for

obvious reasons, and a common practice is to lay a new pipe in parallel with the old one, to which it is joined at several points. It would be best to lay the new pipe all the way from the pump to the reservoir, but it very often happens that there is not enough available money for this and the work is often done piece by piece till it is complete.

If desired, the curves of losses in pipes of Fig. 18 may be used. It is evident that the branched section lets through more water than the single part of the system can handle, and therefore our assumed loss of 12 ft. in the branch is too high and we must try a lower value.

Table V reveals the results of four trials.

TABLE V

Branched Section 6000 ft. long				Single Section 5000 ft. long	
Loss of Head, ft.	Discharge, gal. per min. 18 in. pipe	12 in. pipe	Total gal. per min.	Loss of Head, ft.	Discharge gal. per min.
12	2410	678	3088	136	2650
11	2300	648	2948	137	2660
10	2190	617	2807	138	2670
9	2080	586	2666	139	2680

Suppose that in this case after the pipe has been in service so long that the friction loss has increased 50 per cent for a given flow and a piece of new 18 in. pipe 6,000 ft. long is laid in parallel with the same length of old pipe, to which it is joined at its two ends by Y connections as shown by the dotted line on Fig. 21, what improvement might be expected from this addition? For simplicity in this problem the pressure at the pumphouse is assumed to remain at 300 ft. and the pipe is taken as free from bends and other obstructions, and the loss at the junction points will be taken as negligible compared with the line losses.

This problem is not difficult but requires some thought because of the difficulty of seeing how the water divides at the branch between the old and new pipes, but the former example will prove helpful. In this case the new pipe will greatly increase the capacity of the system, partly because it is new and partly because of its size; in fact, this new piece will considerably reduce the loss between the points it connects. There are various ways of examining the effect of this new pipe, but perhaps the one which will most appeal to the readers of these articles is the old established one of trial and error, a process much shortened by some experience but which can be successfully used by anyone with a little patience.

Of the total loss of 148 ft., a relatively small part will be in the branched section, just as in an alternative route between two cities the time required for a car to travel is much reduced by the two roads that are available. As a first trial we shall suppose that the loss of head in the branched section is 12 ft., then for the 6.000 ft. of new 18 in. with friction factor 0.021 we would have

$$F = 12 = .021 \times \frac{6000}{1.5} \times \frac{V^2}{64.3}$$

from which the velocity will be found to be v=3.03 ft. per sec., corresponding to 2,410 gal. per min. Similarly for the old 12 in. part of the pipe the friction factor is 0:035 (the tabular value increased 50 per cent), and F is 12 ft. as before, from which the calculated velocity in the 12 in. pipe will be found to be 1.92 ft. per sec., and this corresponds to a flow of 678 gal. per min. in the 12 in. pipe.

Now if our assumption of 12 ft. loss in the branched section is right, the 5,000 ft. of single old 12 in. line must carry 2,410 + 678 gal. per min. with a loss of head of 148 — 12 = 136 ft., since the total loss in the system is to be 148 ft. But the discharge just given means a velocity of 9.28 ft. per sec., whereas the loss of head of 136 ft. will occur if the velocity is only 7.51 ft. per sec. as calculated from the relation

$$F = f \frac{1}{d} \times \frac{V^2}{64.3}$$

Evidently the loss in the branched section is practically 9 ft. and the capacity of the system is about 2,680 gal. per min. as compared with a discharge of 2,300 gal. per min. in the original new 12 in. pipe. The small proportionate loss of only about six per cent of the total in the branched section gives ample evidence that the branch should be extended as soon as possible.

SECTION VII

Discharge Through Long Pipe Lines

In the previous section, flow through parallel pipes was considered and the method of finding out the improvement resulting from laying a pipe parallel to a part of a main, to which it was connected at both ends, was explained. In water works distribution systems there are, of course, many such parallel pipes and the mains are so tied together that there are generally a number of paths by which water may travel from one point to another. Ordinarily the system is made up of a complicated network, with many cross connections, and the determination of the conditions of flow in such a network is a very difficult and tedious problem, much beyond the scope of this series of articles. It is, in fact, a problem with which the skilled technical engineer has much difficulty.

One useful problem which does belong to the field being covered but which has not yet been dealt with in this series is that of finding the discharge through a line of pipe made up of several lengths of different sizes of pipe attached end to end. To illustrate this let us suppose that an old 12 in. pipe 2000 ft. long has been extended by adding 800 ft. of new 10 in. pipe, and that this line goes from a pumping station, near which the pressure on the 12 in. pipe is 80 lbs. per sq. in., to a point at elevation 45 ft. above the pumping station, at which point the pressure must not fall below 50 lbs. per sq. in. How much water will the main deliver? This line is shown on Fig. 22, and in calculating the flow it will be assumed that there are no elbows or other sources of loss beyond those due to the straight pipe. The pipes are assumed connected by a tapering reducer in which the loss is comparatively negligible.

In the first section, it was stated that each pound per square inch corresponded to that of a column of water 2.31 ft. high, so that the pressures on the line are 80 × 2.31 = 185 ft. and 50 × 2.31 = 115.5 ft. at the pumping station and discharge ends, respectively. The statement in section IV that the sum of the elevation head, the pressure head, and the velocity head at the first or upstream gage point must equal the sum of the same three heads added to the friction loss between the points, at the downstream gage point will now be applied here, and the datum will be chosen through the gage point near the pumping station. Then $0+185+v_1^2/64.3 = 45+115.5+v_2^2/64.3+F$ and we shall first proceed to find the loss **F** in feet between the gage points. It is given by

$$F = f_1 \times \frac{2000}{12/12} \times \frac{V^2_1}{64.3} + f_2 \times \frac{800}{10/12} \times \frac{V^2_2}{64.3}$$

and in this type of problem it is best to find the relations between the two velocities at once, so that one of them may be written in terms of the other. This is easily done, since for the 12 in. pipe the area of it multiplied by v_1 must equal the area of the 10 in. pipe multiplied by v_2 and, referring to Table I, this means that $0.785\ v_1 = 0.545\ v_2$. Therefore, $v_2 = 1.44\ v_1$ and we may also write $v_2^2 = (1.44)^2\ v_1^2 = 2.074\ v_1^2$.

Looking at Table II of friction factors and taking those corresponding to reasonable velocities, it will be seen that the value of f_1 for new 12 in. pipe is given in the table as 0.022. But as we are dealing with old pipe it will be assumed that the friction has increased 50 per cent above the values given, although the amount of this increase must be estimated by inspection of the pipe or by measurement of the friction in the old pipe. The value 0.033 will be used for f_1 and the table gives a fair average value of 0.022 for f_2, which will be adopted for the new 10 in. piece; closer values for **f** are not necessary in the first calculation as we do not yet know the velocities. Using these values of the friction factors in the formula already set down above gives the relation $F = .033 \times 2000 \times v_1^2/64.3 + .022 \times 960 \times 2.074\ v_1^2/64.3$, and when this is worked out it gives $F = 1.71\ v_1^2$. Putting this value in the first equation above gives $185 + v_1^2/64.3 = 160.5 + 2.074\ v_1^2/64.3 + 1.71\ v_1^2$ from which v_1 is easily found to be 3.78 ft. per sec. and the corresponding value of v_2 is 1.44 v_1 or 5.44 ft. per sec.

This result must now be checked because the values of f had to be taken from the table without a knowledge of the pipe velocities, but it is evident that the chosen values of f happened to correspond with the velocities actually existing. Had they not done so, it would have been necessary to recalculate the pipe with the more correct values of f. For example, suppose we had started with values of 0.03 for f_1 and 0.02 for f_2, then the calculated values would have been 3.95 ft. per sec. in the 12 in. pipe and 5.69 ft. per sec. in the 10 in. pipe and, on referring to Table II for friction factors, we would have found that the value of f for new pipe, corresponding with these values, would have been 0.022 in both cases, and we would have had to repeat the calculation with the revised values of f and would have obtained the same final answer as we got in the first instance. There is usually no way of avoiding the double calculation if accuracy is desired, but experience is helpful in shortening the work in most cases, although even with it the work has often to be done twice over. Some persons may prefer using Fig. 18 for solving this problem, although the writer likes the method given here and often only an approximate result is required.

To get the discharge all that is necessary is to multiply the velocity in the 12 in. pipe by its area, which gives 3.78 × .785 or 2.96 cu. ft. per sec., and on referring to Table I this corresponds to 2.96 × 450 =

1330 gal. per min. The loss of head in friction in the line is $F = 1.71 \times 3.78^2$ or 24.3 ft., and it is easy to show that 14.7 ft. of this is lost in the 12 in. pipe and the remaining 9.6 ft. in the 10 in. pipe. All the hydraulic lines for this case have been drawn on Fig. 22 and some of the pressures are indicated.

Fire Engine Supply Mains

Another application of these principles will be made to a problem which has caused some distress to water works men at times. The difficulty arises during a fire, when a fire engine is drawing water from the distribution system so as to increase the nozzle pressure. Unfortunately, there is most need of the engine if the fire is located in a part of the town where the pressure is low in the main, and this further augments the trouble now being discussed.

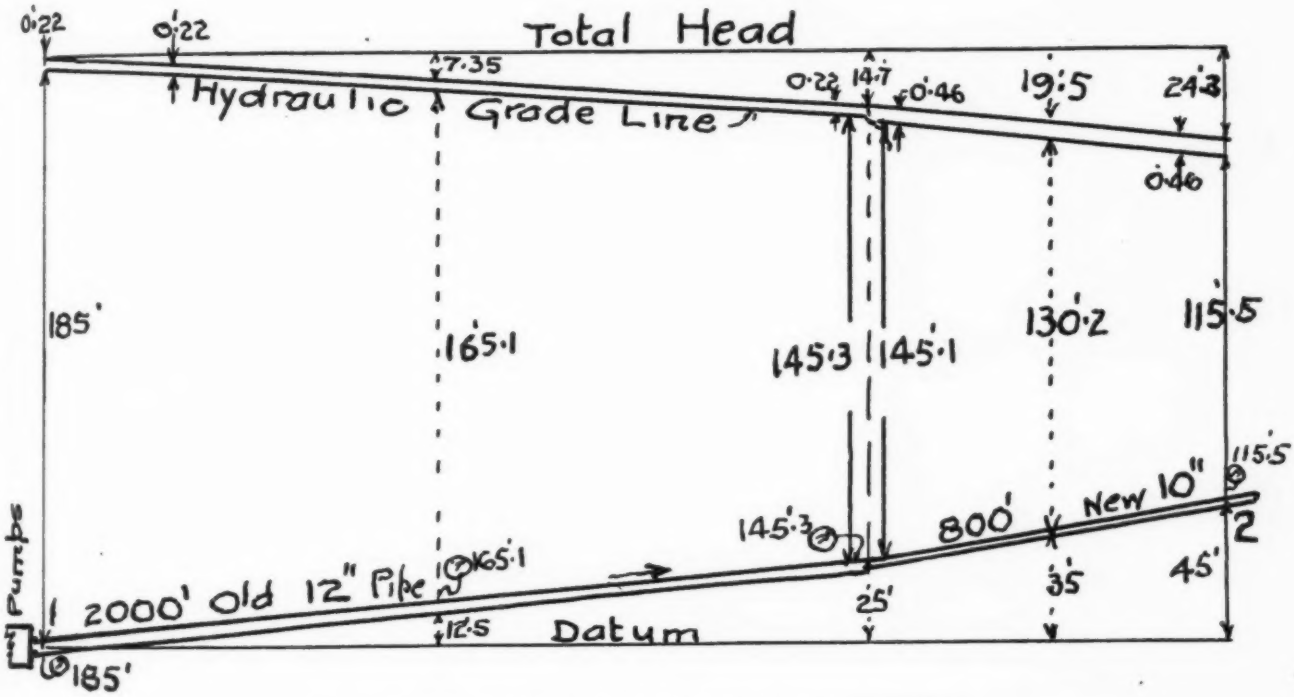


Fig. 22—Discharge Through Long Mains of Pipe of Various Sizes

The sizes of fire engines vary, but let us suppose that this town has one of the larger pumpers having a capacity of 1000 U. S. gal. per minute, and that in dealing with a fire it is attached to a hydrant on a 6 in. water line which is fed from a larger main 600 ft. away. The velocity in the 6 in. pipe necessary to satisfy the full demand of the engine would be 11.33 ft. per sec. (Table I) and this is very high, but we shall examine the case in detail. If the pipe is new, the friction factor **f** is 0.022 (Table II) and the velocity head is 2.0 ft. from Fig. 12, so that the loss of head in friction in the 600 ft. of pipe will be

$$F = .022 \times \frac{600}{6/12} \times 2 = 52.8 \text{ ft.}$$

or practically 23 lbs. per sq. in.

In addition to the loss in the 6 in. pipe there will be a further loss in the hydrant and still more in the connection between the hydrant and the 6 in. pipe. Since hydrants vary a good deal in construction, it is impossible to state the friction losses in them without full knowledge of their characteristics, but with a well made hydrant, through which 1000 gal. per min. is flowing, the friction loss from the 6 in. cast iron pipe to the suction inlet on the engine could hardly be less than 7 ft., and this number will be used here. The total friction loss between the point where the 6 in. pipe leaves the large main and the suction branch of the engine (taken as 6 in. bore) will then be 52.8+7.0=59.8 ft.

Now let us suppose the pressure on the 6 in. pipe close to the large main can be maintained at 25 lbs. per sq. in., or 58 ft. of water, with this draft and that the 6 in. pipe slopes upward 2 ft. in its length (Fig. 23) entering the hydrant at a level 9.5 ft. below the suction branch of the engine. The latter is, therefore, 11.5 ft. above the large main. Taking the datum through the beginning of the 6 in. line, we would find the suction pressure **p** in ft. at the engine from the equation, 0+58+2=11.5+p+2+59.8, from which **p** is found to have a negative value of 13.3 ft., in other words, there is a vacuum of 13.3 ft. at the engine inlet. Obviously the engine must be connected to the hydrant by suction hose that cannot collapse and thus a special suction hose would be necessary.

So far we have proceeded on the assumption that the 6 in. pipe was new, but if it happened to be old and in such internal condition that the friction had an increase of 25 per cent, for instance, above that corresponding to the friction factors in Table II, then, instead of a friction loss of 52.8 ft. in the 6 in. pipe, this will be increased to 66.0 ft. and the vacuum at the engine would be increased by 13.2 ft., bringing it up to 26.5 ft., which is so high that no portable engine would work satisfactorily, even if the suction hose did not collapse. Clearly, under these conditions the 6 in. pipe is too small to supply the 1000 gallon engine and the engine could not work up to capacity unless the pressure in the distribution system could be raised.

We might now see what advantage there would be in using an 8 in. pipe instead of the 6 in. For an 8 in. pipe the velocity necessary to deliver 1000 gal. per min. would be 6.37 ft. per sec. (Table I), which is still high, but the resulting friction loss in the pipe would only be 13.0 ft. (Table II) if the pipe was new, instead of the 52.8 ft. loss in the smaller pipe. With the larger pipe, even when it is old enough to show an increase in friction of 25 per cent above that for new pipe, there will be a suction *pressure* of 21.5 ft. at the engine, which is more than required. This problem illustrates the decided advantage of the larger line. As a matter of fact, a 6 in. line is too small to supply fire hydrants in most places.

Overflow Pipe in a Tank

We shall next turn attention to the tank with overflow pipe as shown in Fig. 24, the dimensions being indicated there, and we shall assume the pipe to be new and 4 in. inside diameter, although if steel pipe is used it will be slightly larger. The discharge is freely into the atmosphere and we wish to find how much water this overflow can carry away if the water in the tank is not to rise over 2 ft. above the top of the pipe. There are three ways in common use of shaping the inlet end of the pipe and these are shown at (a), (b) and (c) on Fig. 24. For the case (a) no special device has been used at the pipe end except to cut it off square at the end and remove any burrs, but the edges have not been rounded in any way. For the case (b) a flat plate about 15 in. diameter has been fitted to the pipe so that the top of the plate is just flush with the top of the pipe, an arrangement sometimes referred to as the "standard end." In the case (c) a bell mouth has been attached to the top of the pipe, which, of course, involves some expense.

We shall now see what effect these ends have on the discharge, and since the resistance at entrance is not small compared with the other losses, we shall approach the matter in a somewhat different way to that of using the entrance losses as given in Table III, although that method may be used if desired and will give substantially the same results as the one now to be used. Substituting an equivalent length of straight pipe for an entrance loss is a little more accurate where the

Fig. 23—Example of a Fire Pumper Attempting to Take a Full Supply from a 6 in. Line

pipes are long than where they are short. Many experiments have shown that the entrance loss in case (a) is 80 per cent of the velocity head in the pipe, for case (b) it is only 50 per cent of the velocity head, while for case (c) it falls to only 10 to 20 per cent, depending on the shape and finish of the bell, and we shall use 15 per cent in this example.

Following the method already laid down, of adding the elevation, pressure and velocity heads at the first section and equating this sum to the sum of the same three heads at the second section plus the friction head lost between the sections, we shall choose the first section at the surface of the water in the tank and the second section at the discharge end of the pipe, and the datum will be chosen through the second section. For the first section we have only the elevation head of 26 ft., since the pressure and velocity at that point are each zero, so that $26 = 0 + 0 + v_2^2/64.3 + F$, since both the pressure head and the elevation heads at section 2 are zero.

The friction head **F** is the sum of the loss at entry and that in the straight part of the pipe and for the entrance (a) this becomes $(0.8 + f\ 1/d)\ v_2^2/64.3$. The value of **f** is taken from Table II for a high velocity since the velocity in the pipe is likely to be large. A suitable value is probably 0.022. Further, **l** is 24 ft. and **d** is 4/12 ft., and from these the value of **f l/d** is 1.58. Putting this value in the last equation in the paragraph above gives $26 = (0.8 + 2.58)v_2^2/64.3$ for the case with the entry end (a), from which the pipe velocity is easily found to be 22.2 ft. per sec. which corresponds to a discharge of 870 gal. per min. as found from Table I. Putting in the entry values 0.5 and 0.15 (50 per cent and 15 per cent) for entrances (b) and (c), respectively, results in a calculated discharge of 913 gal. per min. (velocity 23.3 ft. per sec.) for entrance (b) and 957 gal. per min. for the bell mouth entrance, this corresponding to a velocity of 24.4 ft. per sec.

It is easy to show that a gage attached to the pipe at the base of the tank will show a vacuum of 5.9 ft. so that if the lower end of the pipe is cut off, the discharge will decrease. To put this into definite figures let us suppose that the pipe is cut off one foot below the bottom of the tank, reducing its length to 13 ft. and bringing the original head of 26 ft. down to 15 ft., then it will be found that the discharges are reduced to 747 gal. per min. for the end (a), 792 gal. per min. for the end (b), and to 860 gal. per min. for the bell mouth end, reductions which are considerable. The end condition evidently has a marked effect on the discharge and the expense involved in improving this condition is often not serious. There may be a slight error in the friction factor as the table does not go up to these high velocities, but the results are close to the correct ones.

SECTION VIII

Design of Discharge Openings in Tanks and Reservoirs

Inasmuch as we have been discussing an overflow, and incidentally the effect of the entry end on the discharge, this is probably the best place to deal with the drainage outlet openings for tanks and reservoirs and the best design for them. Only round openings are considered, as this is the most usual shape.

Liquids may be discharged from tanks through various types of openings, and following the order used in the article discussing the overflow, we may present these openings, illustrated in Fig. 25. First there is the short piece of smooth pipe which has not been well fitted into the bottom but projects up into the tank as shown at (a). At (b) in Fig. 25 is shown an opening of the same character but carefully fitted so that the top of the pipe is just flush with the inside bottom of the tank. In both of these cases the burrs are supposed to be removed from the inside of the pipes, and the ends of them are cut off smoothly and square, but there has been no rounding of any of the corners.

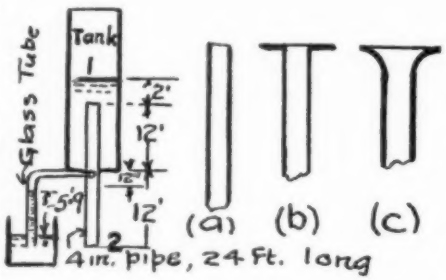


Fig. 24—Discharge Capacities of Tank Overflow Pipes of Various Inlet Designs

A better construction is the bell-mouth entrance shown at (c). This need not be expensive construction, although its efficiency depends on the care taken in forming the bell-mouth and in making it smooth. It is sometimes made of only the part above the line **BB**, and sometimes the pipe below is added, so we shall examine both cases later. It is usually set flush with the bottom of the tank, as shown in the drawing, but it is almost as effective if used on top of the design (a) or (b), in which case it would, of course, project up into the tank. The orifice (d) is the cheapest and, unfortunately, is very common; it consists of a smooth circular hole bored in the bottom of the tank, with the burrs around it removed but with the corners left sharp. The orifice has smaller capacity than any of the other designs shown.

For the designs (a) and (b) the effectiveness is almost completely lost unless the length of the pipe is from three to four times its internal diameter, whereas the bell-mouth (c) has no such limitations. The reason for this is that in the first two cases the water first suffers a marked contraction near the top of the pipe, but if the pipe is long enough the stream will again expand and completely fill the discharge end in the way shown on the drawing.

For the orifice (d) the stream contracts until its area is almost 62 per cent of that of the orifice, which means that the diameter of the stream is 79 per cent of that of the opening. The stream bears the classical name of the *vena contracta*, or contracted vein, and has been the subject of very many studies and experiments. The minimum diameter of the vein is reached at a distance of about half the orifice diameter away from the face of it and the stream then remains uniform in size till the air resistance breaks it up. When the orifice is in the vertical side of a tank and the water in the latter is clean and free from turbulence, the stream looks like a clear glass bar, being fixed in position but gradually bending downward in a graceful curve under the action of gravity.

The frictional losses in the stream from the orifice are only about three per cent of the head of water above the center of the orifice or three per cent of the velocity head in the contracted stream. It is evident that if a gage could be placed in the stream in such a way as to measure the pressures, then at the orifice the gage would read higher than it would in the contracted vein, since the decrease in area produces an increase in velocity which will correspond to a fall in pressure. The remaining arrangement, shown at (e) in Fig. 25, is made up by attaching a conical end, instead of a uniform pipe at **BB** below the mouthpiece shown at (c), but the angle at the cone must not much exceed about six degrees or the stream will not expand fast enough to keep the cone full and the stream probably will pass through as shown at (f), where the conical extension might just as well be dispensed with. The cone shown at (e) has an angle of five degrees between its side and will, therefore, require a length of 69 inches to give it an outlet diameter of 12 inches. The outlet end must be submerged for effective action.

An effort has been made to show a similar set-up for each design, but this is not entirely possible, although the comparative merits of the different designs will be clear. In each case it is assumed that the depth of water over the entrance is sufficient to prevent air from being drawn from the water surface down through the center of the opening, and for the cases shown this depth has been taken as 2 ft., although it could be much less without causing trouble. The discharge pool has been taken at 2 ft. below the tank, but this is only of interest in connection with design (e) as the surface only needs to be clear of the tubes in the other cases. The minimum diameter is 6 in. in each case. We have already examined three of these designs in connection with the discussion of overflows so that it is not necessary to discuss them at great length.

Inward Projecting Pipe

For this form, shown at (a) Fig. 25, the entrance loss has already been given as about 80 per cent of the velocity head, which includes the entire length of this short pipe. Taking the datum at the water surface in the pool, we may write the elevation head of 4 ft. of a pound of water at the surface in the tank, where there is neither pressure nor velocity, as equal to the velocity head at exit from the pipe, plus the loss, a statement which is true because there is no pressure at the exit end and the elevation head is evidently zero. We may, therefore, write:

$$4 = \frac{V^2}{64.3} + \frac{0.8\ V^2}{64.3}$$

from which **V**, the velocity at the discharge end, is easily found to be 11.95 ft. per sec., and Table I shows this to correspond to a discharge of 1055 U. S. gal. per min.

Standard End

For the design shown in Fig. 25 (b) and often referred to as the standard end, the equation may similarly be written:

$$4 = \frac{V^2}{64.3} + \frac{0.5\ V^2}{64.3}$$

since the loss, in this case, has already been given as half the velocity head at the discharge end, and from this the value of **V** is 13.1 ft. per sec., which means a flow of 1155 gal. per min., and this is practically 10 per cent higher than in the former case.

Bell-Mouth End

It is a little difficult to make a fair comparison of the bell-mouth end shown in Fig. 25 (c) with the former pair because there is not the same inherent reason for having the pipe below the line **BB**. Let us then make two calculations, the first with only the mouth-piece, the pipe below **BB** being omitted, and the second with this pipe being included. Where the mouth-piece alone, without the extension, is used, the loss has already been given as 10 to 20 per cent of the velocity head and we shall assume that the work is so smooth that the lower value will apply. Then

$$2.25 = \frac{V^2}{64.3} + \frac{0.10\ V^2}{64.3}$$

from which the discharge velocity is found to be 11.45 ft. per sec. and the discharge 1010 gal. per min., the least of the three.

Let us now see the effect of adding 21 in. of uniform size pipe below the line **BB**, attached with water tight joint there and with the lower end of it level with the pipes in the cases (a) and (b) as shown at (c) Fig. 25. In this case we must add the loss in this 21 in. length, small though it is, because the entry loss does not include it. In the cases (a) and (b) the entrance loss includes a length of four pipe diameters and in figuring the overflows we should have made allowance for this, but in longer pipes its effect is so small that it is generally neglected. For this short pipe, however, each loss must be carefully included and that loss in this 21 in. of 6 in. pipe will be given by the formula

$$\frac{fl}{d} \times \frac{V^2}{64.3}$$

where the value of f is close to 0.022, from Table II, and **l** is 1.75 ft., and since d is 0.50 ft., the loss in this piece works out to 0.08 times the velocity head. Writing out the equation for the combined mouth-piece and pipe gives

$$4 = \frac{(0.1 + 0.08 + 1)\ V^2}{64.3}$$

or the discharge velocity is 14.75 ft. per sec. and the discharge of 1300 gal. per min. is a decided improvement over the former designs. The cost of it is not necessarily high.

Circular Orifice

If the discharge opening is just a hole 6 in. in diameter bored through the bottom plate of the tank and the burrs are removed, but the corners are not rounded, the stream will take the circular orifice form shown at (d). There is a contraction of the jet, as stated before, and the minimum diameter of 6 × 0.79 or 4.74 in. is reached about 3 in. below the inside bottom of the tank. The pressure in the jet is atmospheric just where the diameter of it becomes constant and is at the place shown in Fig. 25 (d). The loss up to this point is equal to

$$\frac{0.03\ V^2}{64.3}$$

V is the velocity in the smallest part of the stream, where the diameter is only 4.74 in. and the area 0.122 sq. ft. Following the method used for the other designs it is possible to write

$$2.25 = \frac{V^2}{64.3} + \frac{0.03\ V^2}{64.3}$$

which gives the velocity in the contracted jet as v = 11.85 ft. per sec. The discharge is, of course, found by multiplying this velocity by the area 0.122 sq. ft., which gives 1.446 cu. ft. per sec., or 650 gal. per min., which is the smallest of all. This is rather curious when the small loss is considered, but the reduction in the area of the stream more than counteracts its other advantages.

Effect of the Conical Piece

We have already seen the advantage of adding a piece of uniform sized pipe below the mouth-piece and shall now study the effect of adding a conical piece so as to make the shape shown at (e) Fig. 25. If this is to be effective then, as already stated, the angle of the cone must be small enough so that the water will expand at a sufficient rate to keep the pipe full from end to end, for if the cone angle is too great, the active part of the stream of water will run down the center of the cone as shown at (f) in Fig. 25, in which case the cone is of little service. An angle of five degrees between the sides is shown here at (e), and if the outlet diameter of the cone is 12 in., the length of it must be 69 in., but if this is too long to place vertically, little harm will be

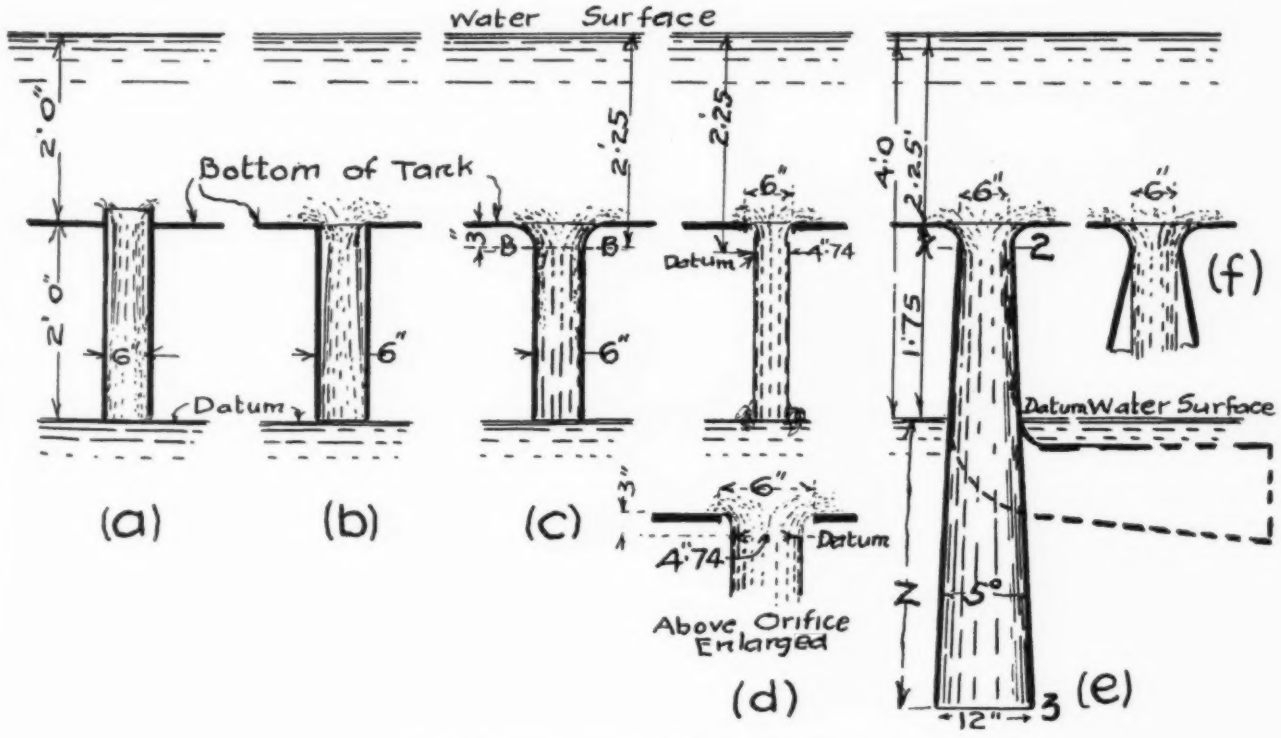


Fig. 25. Various Types of Discharge Openings
(Top edges in (a), (b), and (d) are sharp but with burrs removed.—Pipes and orifices are circular.)

done by constructing it with a right-angled bend, as shown dotted. In either case care must be taken to see that the discharge end is fully submerged.

Taking the datum at the lower water surface, we have

$$4 = -z + z + \frac{V_2^2}{64.3} + F$$

and in calculating **F** we notice that the whole tube is made up of the bell-mouth entrance, in which we have already taken the loss as

$$\frac{0.10\ V_2^2}{64.3}$$

and the conical part, in which the loss should not exceed

$$\frac{0.04\ V_2^2}{64.3}$$

and these two together give the total value of **F** as

$$\frac{0.14\ V_2^2}{64.3}$$

Inasmuch as the throat diameter is half that at the outlet, the corresponding area is one-quarter of the outlet area, and the throat velocity will be four times that at the outlet, that is, $V_2 = 4V_3$, and also, of course, $V_2^2 = 16V_3^2$. Placing these values in the equation

$$4 = \frac{V_3^2}{64.3} + F$$

gives

$$4 = \frac{1}{16} \times \frac{V_2^2}{64.3} + \frac{0.14\ V_2^2}{64.3}$$

from which the throat velocity V_2 is found to be 35.6 ft. per sec., and the corresponding discharge is 3140 gal. per min. This much exceeds that from any of the other designs.

The conical extension is, therefore, most valuable and the reason will become clear if we calculate the pressure at the throat. Taking the datum at the lower water surface, we have

$$4 = 1.75 + p + \frac{V_2^2}{64.3} + F$$

and since V_2 is 35.6 ft. per sec., then $\frac{V_2^2}{64.3}$ is 19.7 ft. and the value of F is 0.10×19.7 or 1.97 ft., which is the loss down to the throat. Hence, $4 = 1.75 + p + 19.7 + 1.97$, or p is — 19.42 ft., showing that there is a vacuum of 19.42 ft. at the throat. Therefore, with the expanding cone the pressure forcing the water through the bell-mouth part is 2.25 ft., and the water is above the throat, plus the suction of 19.42 ft., giving a total head on this part of 21.67 ft. as compared with only 2.25 ft. if the cone is not attached. Examination of the method of calculation will show that for this mouthpiece the discharge varies with the square root of the head and hence

$$\frac{\text{Discharge with conical extension}}{\text{Discharge without extension}} = \sqrt{\frac{21.67}{2.25}} = 3.105,$$

which is just the ratio of the two discharges 3140 and 1010 gal. per min. which we found.

If we assemble the results of this study, they will be found in Table VI.

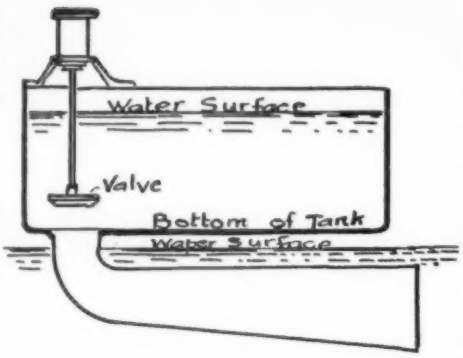


Fig. 26. Tank Fitted with Bell-Mouth and Conical Expansion Extension
(Valve operated by hydraulic cylinder)

TABLE VI

Design See Fig. 25	Inward Projecting (a)	Standard End (b)	Bell-Mouth Entrance (c) No Pipe Attached	Bell-Mouth Entrance (c) With Pipe Attached	Orifice (d)	Bell-Mouth with Cone (e)
Discharge U. S. gal. per min.....	1055	1155	1010	1300	650	3140

This study shows that with the same sized 6 in. opening in each case the capacity to carry away water varies enormously with the design. Evidently the time required to empty the tank may be greatly reduced by careful design of the outlet and in cases of old tanks, where it is necessary to speed up the process and it is not possible to increase the opening in the tank, then some of the designs examined may be useful. It should be noted that in cases of designs (a) and (b) if the length of the pipe is less than about four diameters, the expanding water will not fill the bottom end of the tube and the net effect will be the same as if the orifice shown at (d) is used. The conical tube is very helpful and has been successfully used, but it must be remembered that when the tank is nearly empty, air will be drawn down from the water surface in all of these designs and the method of calculation does not hold good then.

If it is desired to speed up the discharge from a tank with an orifice (d) without enlarging the hole, obviously this may be done by placing a bell-mouth above it and a conical piece below it, making sure that the inside surface is left smooth and clean. If the discharge opening is to be closed and opened, some type of valve must be used on it, and obviously any ordinary type of gate valve would offer a great deal of resistance and very largely nullify the effect of an otherwise good design. In a number of instances the writer has used flat disks with rubber faces and so designed that they may be lifted vertically away from their seats, thus leaving the opening free for the water to enter. The disk may be lifted by lever or by means of a hydraulic cylinder, and a diagram of the latter type applied to an opening with an expanding cone is shown in Fig. 26.

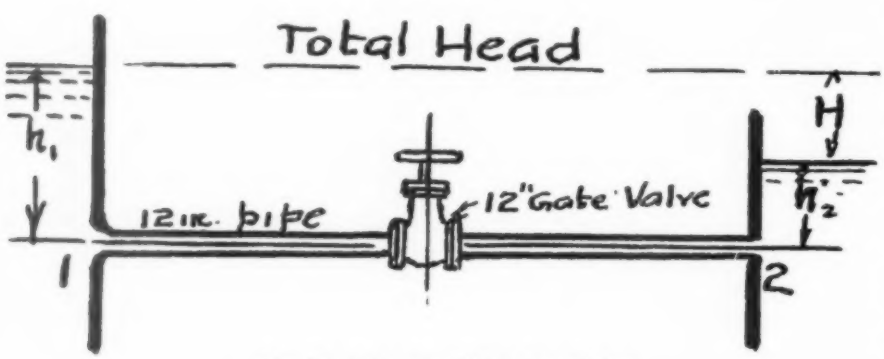


Fig. 27. 12-in. Pipe With Gate Valve

SECTION IX

Effect of Valves in Pipes; Flow Through Fire and Metering Nozzles

To further illustrate the great number and the variety of problems that may be solved by the principles laid down, the action of valves in pipes may be examined. As ordinarily made, valves produce two effects, for they control the area of the waterway through them and they also produce relatively large losses of head, particularly as they approach the closed position. Reduction in the area of the waterway alone is not a very effective way to control the flow.

Valve Experiments

Valves are commonly of either the gate or the globe type, the former being very largely used on medium or very large water pipes and the latter on smaller ones. There is no fixed rule in this matter. The loss of head through globe valves is higher than that through gate valves under similar conditions, as an examination of their constructions would suggest, but the data are much more complete for the gate valves than for the others. Probably the best authority on this subject is the excellent booklet issued in 1922 by the Engineering Experiment Station of the University of Wisconsin, Madison, Wis., which records an exhaustive investigation by Professor C. I. Corp under the title, "Experiments on the Loss of Head in Valves and Pipes of One-half to Twelve Inches Diameter." The largest globe valve tested was two inch.

This is not the place to enter into a detailed discussion of this work and we must be content with such abstracts from it as will suit our present purposes. Realizing that the loss of head in valves, as well as in most other fittings and pipes through which water flows, bears a relation to the velocity head in the pipe to which they are attached, the experiments were used to give, amongst other things, a coefficient **"k"** for each size of valve and for different openings of each valve, the coefficient being such that the loss of head in the valve for the given opening is

$$\frac{KV^2}{64.3}$$

where **v** is the velocity in the pipe.

Crane and Chapman valves were mainly used and the valve opening is designated

by the height of the opening under the gate relative to the diameter of the full opening under the gate, or quite closely it is the number of turns of the stem after the waterway begins to open relative to the number of turns required for full opening. (In most valves the stem must be turned some before the waterway actually begins to open and these idle turns are not counted.) For 12 in. gate valves the following values of **k** were taken from Professor Corp's curves:

For 12-in. Gate Valves—Values of K in $KV^2/64.3$

giving the loss of head in feet, where V is the pipe velocity in ft. per sec.

Valve Opening	Value of K
Full	0.07
¾	0.41
½	2.25
¼	13
⅛	56

For a 2 in. globe valve, fully open, **k** is 7.1 as compared with 0.16 for the same size gate valve when fully open. The very high relative loss in the globe valve is due to the tortuous passages for the water.

Effect of Closing a Valve In a Pipe Line

To show the use of these numbers, and also to obtain an intelligent view of how a valve acts, we shall take two examples, the one of a 12 in. new pipe, 100 ft. long, between two reservoirs and with a 12 in. gate valve in the line. Inasmuch as it makes no difference where the valve is so long as it is not very close to the outlet end, it will be assumed to be placed in the center of the line. The action of closing the valve in this case will be compared with what happens in a similar operation where the valve is in a 12 in. pipe 1000 ft. long and old enough to show an increase of 20 per cent above the friction factors of Table II.

The illustration in Fig. 27 covers both cases, and in order to make the comparison as fair as possible, approximately the same velocity is assumed to exist in both pipes when the valve is fully open. For this reason the head **H** is 2 ft. for the short pipe and 17 ft. for the long one.

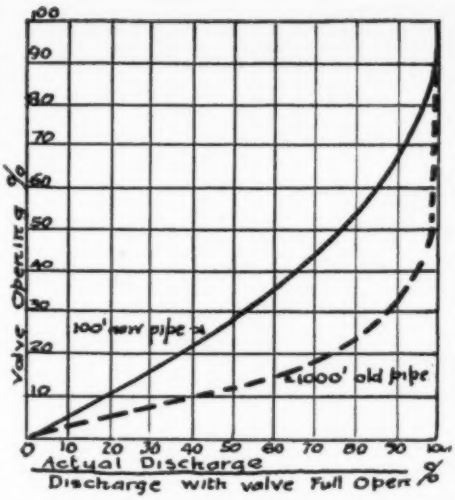


Fig. 28. Effect of Closing the Valve in the Line Shown in Fig. 27

The pipes are horizontal and the datum is selected along the axis so that all elevation heads on the pipe are zero. Starting with a pound of water at the surface of the left hand reservoir, Fig. 27, where there is neither pressure nor velocity, the total head is evidently h_1 and if the conditions at the right hand reservoir, just where the water leaves the pipe, are examined it is seen that the pressure head is h_2 and the velocity head is

$$\frac{V^2}{64.3}$$

where **v** is the velocity in the pipe. Hence

$$h_1 = h_2 + \frac{V^2}{64.3} + F$$

Now **F** is the friction head lost in the pipe and valve, and since the pipe is shown with bell-mouth entry the loss at that end of the pipe is small enough to be neglected. The total loss **F** is then

$$\frac{fl}{d}\frac{V^2}{64.3} + \frac{KV^2}{64.3}$$

and by placing these values in the equation above there results

$$h_1 - h_2 = H = \left(1 + K + \frac{fl}{d}\right)\frac{V^2}{64.3}$$

As the velocity is likely to be less than 5 ft. per sec. the value of **f** from Table II will lie between 0.021 and 0.025 for the new pipe, but will be 20 per cent higher for the old pipe and will, therefore, lie between 0.0252 and 0.03 in the latter case. It will be necessary to make preliminary estimates of **f** for the first trial, but it may be necessary to repeat the calculations with better values of **f** once the approximate velocity is known.

Starting with the shorter pipe and the valve wide open, we write

$$2 = \left(1 + 0.07 + \frac{0.021 \times 100}{1}\right)\frac{V^2}{64.3}$$

or—

$$\frac{V^2}{64.3}$$

is found to be 0.631 ft. and the corresponding pipe velocity **v** is 6.37 ft. per sec. With the same pipe and the valve three-fourths open, the only change in the equation above is that 0.07 will be replaced by 0.41 and consequently the pipe velocity will be 6.05 ft. per sec. Similarly, for the half open valve **v** is 4.89 ft. per sec. while for the one-quarter open position it is 2.81 ft. per sec., and for the one-eighth open position it is 1.47 ft. per sec. (For the last case the value 0.025 was used for **f**.)

Long Pipe with Valve Being Closed

Before discussing these results, the corresponding figures for the long pipe will be obtained, for which case the head **H** is taken as 17 ft. so as to give approximately the same initial pipe velocity as was used for the shorter pipe. For the full open valve we have

$$17 = (1 + 0.07 + 25.2)\frac{V^2}{64.3}$$

from which the pipe velocity is 6.45 ft. per sec. For the ¾, ½, ¼ and ⅛ valve openings the values of the pipe velocity are found to be 6.41, 6.20, 5.28, and 3.62 ft. per sec., respectively.

For convenience the results are set out in the table on page R-32.

These figures are plotted on Fig. 28, on which the discharge for each valve opening is given in percentage of that with the valve fully open. Obviously with the short pipe, in which the pipe resistance is generally much smaller than in the valve, the effect of valve closing is evident from the

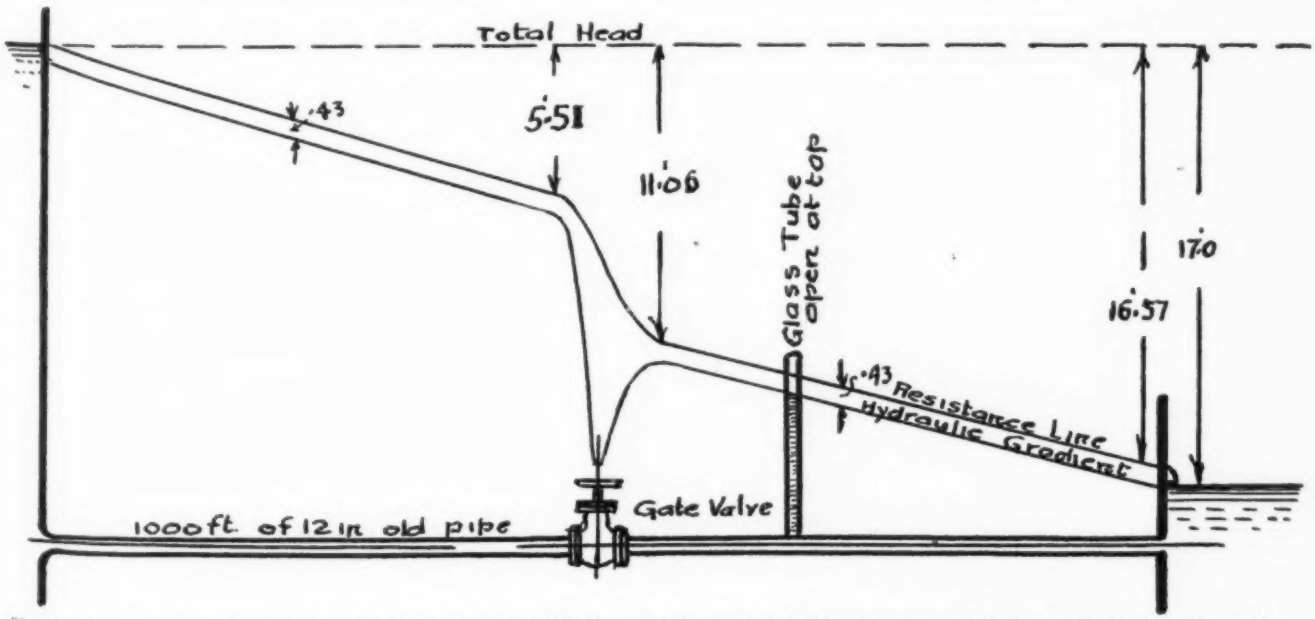


Fig. 29. Hydraulic Lines for 1000 ft. of Old 12-in. Pipe With Valve ¼ Open (Note how the pressure falls precipitately at the valve)

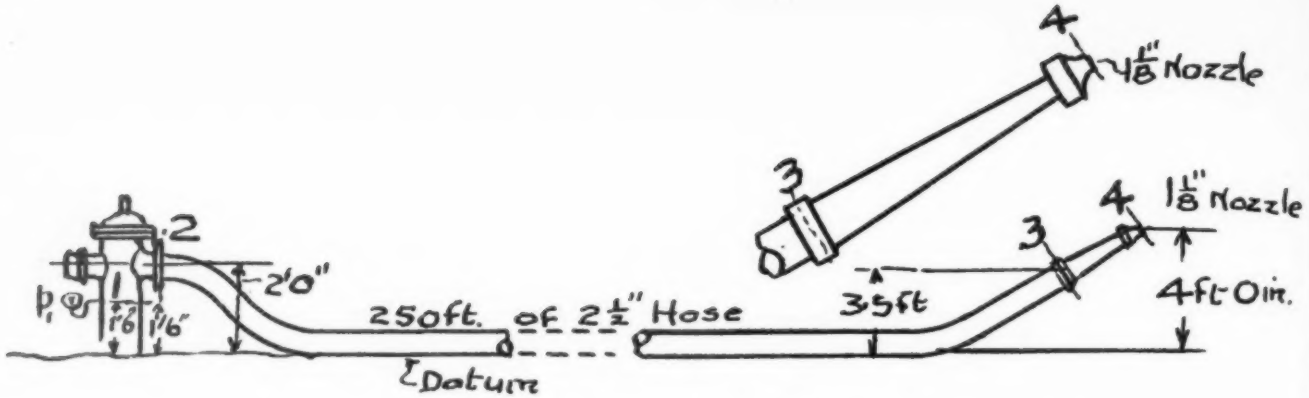


Fig. 30. Fire Stream from a Hydrant

beginning of the operation while for the long pipe it is fairly accurate to say that the first part of the closing has little effect, for when the valve has been closed half way the discharge is still over 96 per cent of that for the full open position. The hydraulic lines for the long pipe with the valve three-quarters closed are shown on Fig. 29 and attention is called to low pressure in the valve.

Flow Through Nozzles

Only one other illustration of the flow in closed pipes will be given and that is in connection with the nozzle, a very common application of which is in connection with fire streams. Fire nozzles are made of various sizes and shapes, but attention will be confined to the smooth parallel type. The nozzles are threaded so that the firemen may conveniently attach any desired one to the play pipe, the latter being permanently attached to the fire hose (usually 2½ in.) and which is connected to the hydrant or fire engine. As a general thing, a single line of hose is used for a single nozzle but for bad fires, requiring a lot of water and a strong stream, two or even three lines of 2½ in. hose may be coupled in parallel to the engine and joined at the outlet end by a Siamese connection to a short piece of larger hose with play pipe on which a single large nozzle may be used.

It is not the job of the water works engineer to specify the requirements for fire fighting; that is done by the Board of Fire Underwriters, who give for each size of town the number of fire streams of given size, and attached to a specified length of 2½ in. hose, which must be available at any point where a fire occurs. For instance, in Canada they require that in a town of 10,000 inhabitants provision is to be made to supply eight standard fire streams, each of which must be able to deliver 250 U. S. gallons per minute through a 1⅛ in. nozzle on the end of a single line of 2½ in. rubber-lined fire hose 250 ft. long, and it is our concern to find the hydrant pressure necessary for this effect and also to find the size of main necessary.

A hydrant with one such stream is shown on Fig. 30 and an enlarged view of the play pipe and nozzle is also shown. The area of the 1⅛ in. nozzle is 0.00691 sq. ft. and for the discharge of the 250 gal. per min. the velocity is 80.5 ft. per sec. with a corresponding velocity head of 101 ft., while for the 2½ in. hose the numbers are 16.4 per sec. and 4.18 ft. (Table I and Fig. 12 will be found helpful in getting these results.)

Taking Sections 1, 2, 3 and 4 at the points shown on the drawing in Fig. 30, it will be assumed that the barrel of the hydrant is so large that the velocity head in it is relatively negligible. (For instance, if the body of the hydrant has a net area equivalent to that of a pipe 5 in. in diameter, the velocity in it will be only 4.1 ft. per sec. for which the velocity head is only 0.25 ft., which is relatively insignificant in this problem.) It is also fair to assume that the hydrant is well made and that the entrance to the outlet where the hose is attached offers little source of loss.

	Valve Opening	Full	¾	½	¼	⅛
Short Pipe H = 2 ft.	Pipe velocity, ft. per sec.	6.37	6.05	4.89	2.81	1.47
	Flow, gal. per min.	2248	2135	1726	992	519
	Percent full open flow	100	95.0	76.8	44.1	23.1
	Valve Opening	**Full**	**¾**	**½**	**¼**	**⅛**
Long Pipe H = 17 ft.	Pipe velocity, ft. per sec.	6.45	6.41	6.20	5.28	3.62
	Flow, gal. per min.	2275	2260	2190	1850	1277
	Percent full open flow	100	99.4	96.4	81.4	56.2

Applying the principles already established gives the equation

$$1.5 + p_1 = 2 + p_2 + 4.18$$
$$= 3.5 + p_3 + 4.18 + F \text{ (up to 3)}$$
$$= 4 + 101 + F \text{ (in hose \& nozzle)}$$

The friction loss up to Section 3, that is in the hose, is

$$f\frac{l}{d} \times 4.18$$

where **l** is 250 ft. and **d** is 0.208 ft., which makes this loss 5024.f ft. The friction factor **f** for iron pipe of this size and velocity is given as close to 0.022 in Table II, but best grade rubber-lined hose is undoubtedly smoother than the iron pipe and it would probably be quite accurate to take a value of **f** at 15 per cent lower than the tabular value; 0.0184 will, therefore, be used. This will give the loss in the hose alone as $0.0184 \times 5024 = 92.3$ ft.

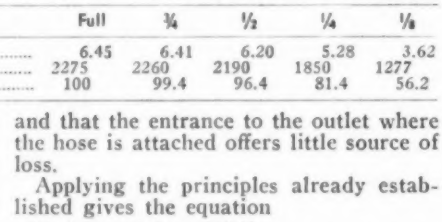


Fig. 31. Nozzle Arranged for Measuring Discharge

Many experiments on the fireman's play pipe and nozzle show that for the sizes ordinarily used with single lines of hose the loss of head in the play pipe and nozzle is almost exactly equal to the velocity head in the hose, which is 4.18 ft. in this case. The total friction loss, then, from the hydrant to the discharge end of the nozzle is 92.3+4.18 or 96.5 ft. To get the hydrant pressure p_1 it is only necessary to write the equation in the form

$$1.5 + p_1 = 4 + 101 + F = 4 + 101 + 96.5,$$

or the hydrant pressure p_1 is 200 ft. or 86 lb. per sq. in. Since the velocity head in the jet is 101 ft., that is the theoretical height of the highest drops if the stream is directed vertically upward in still air, some of them would go nearly that high, but as an effective fire stream it would not reach over 70 ft. in height.

Where the nozzle is used for other purposes the play pipe is not required and the nozzle is frequently fastened to the end of a pipe or inserted somewhere along its length as shown on Fig. 31 for the first case and if carefully made with well rounded entrance the loss in it is very small indeed. It is frequently used for measurement of discharge, but if great accuracy is desired, the shape of the nozzle should be that used by the American Society of Mechanical Engineers or some other authority, where the coefficients have been determined by experiment. Such nozzles have a cylindrical exit end about one diameter long, but consideration of this use of the nozzle will be left for the next article.

Often small nozzles are used on large pipes or tanks to produce jets for fountains or for other purposes such as sprays. If a definite solid stream is desired, the nozzle must be smooth and with well-rounded entrance, in which case the velocity of the stream leaving the nozzle is about 97 per cent of the theoretical value. Thus if a one-half inch nozzle is attached to the side or top of a large pipe or tank in which the pressure, corrected to the level of the nozzle, is 10 lb. per sq. in. or 23.1 ft. of water as shown by gage, the velocity of the water where it leaves the nozzle will be approximately $0.97\sqrt{64.3 \times 23.1}$ or 37.4 ft. per sec., and as the area of the half-inch opening is 0.00136 sq. ft., the discharge will be $0.00136 \times 37.4 \times 450$ or 22.9 gal. per min., since one cubic foot per second is the same as 450 gal. per min. The jet should rise well over 20 ft. unless there is a wind to break it up.

Where it is desired to make a spray instead of a solid jet, the shape of the nozzle must be much modified and must be designed by experiment.

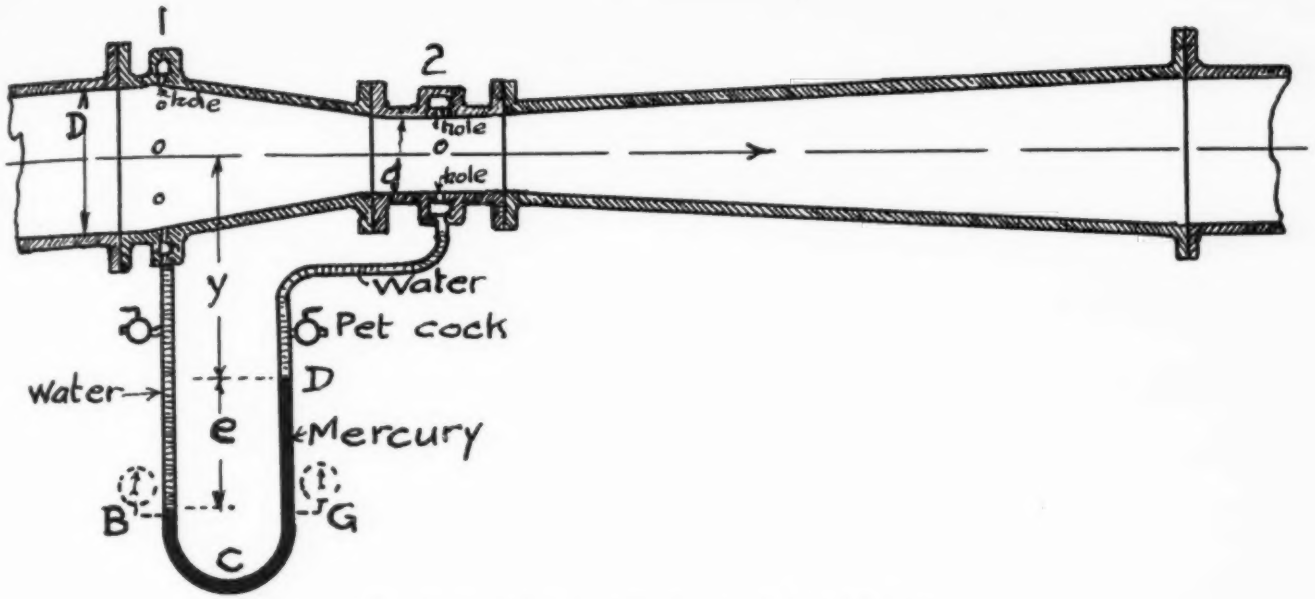


Fig. 32. Venturi Meter with Mercury Differential Gage

SECTION X

Measurement of Discharge Through Pipes

Under this heading only pipes running full of liquid, or *closed pipes* as they are called, will be discussed and the treatment of pipes flowing partly full, as in ordinary sewers, drains, etc. will be deferred to a later article.

For all small pipes and for a good many large ones, meters of the disc, helical, compound or other similar type are in common use and are familiar to the water works man, so that they need not be described here. The construction of them has more to do with mechanical problems than with hydraulics. Such meters must, of course, be calibrated from time to time, and the loss of head checked, and this should be routine practice in water works operation. There are, however, many metering devices that depend upon the hydraulic principles considered in these articles and some of these devices will now be given attention.

The Venturi Meter

The nozzle was referred to in the last article, but it would be better to start with what is probably the best known measuring device; that is the Venturi meter, so generally used on main lines and in pumping stations, where the output of the pumps is measured and recorded. This meter was mentioned in Section I and its construction is well known to consist of two conical pieces with a short cylindrical piece between them—(Fig. 32). It simply forms part of the pipe line, and its outstanding feature is that it offers extremely little resistance to the flow.

The up-stream cone has its inlet end the full pipe diameter and tapers down fairly rapidly to the size of the cylindrical throat piece attached to it. The cylindrical throat is, in turn, coupled to an expanding cone, of rather small taper, and with its outlet end the full size of the pipe again. The cones are usually smooth castings but the water passages are not machined, while on the other hand the throat piece is bored smooth and to an exact size and its connections to the two conical pieces are carefully rounded off so that the whole meter will offer the least possible resistance to the flow.

The outline of such a meter with cones of a usual taper is shown in Fig. 32. As illustrated, it is used in a horizontal pipe, which is the usual practice, although the Venturi tube may be placed in vertical pipes or those sloping at any angle. As these meters are made by different builders, their proportions vary considerably. Since the discharge through any particular size of Venturi tube depends directly on the extent of the drop in pressure ($H = p_1 - p_2$) in feet between the entering end of the upstream cone and the lesser pressure at the throat, builders of the meter have devoted much thought and care to the accurate measurement of this pressure drop.

Hollow rings surround the meter at the pressure measuring points, as shown on the figure, and a number of holes are drilled so as to connect the inside of these rings with the inside of the meter, thus insuring that the pressure on the inside of each ring is an average of all the pressures around the meter wall at these particular points. Much study has been given to the design of these pressure holes which must not be large and must have the burrs removed from the inner end of them, yet without appreciable rounding of their inner edges.

Theory of the Venturi Meter

The theory of the meter is very simple. Taking the horizontal one illustrated, the datum may be assumed along the axis of the tube, and if we write down the different heads, using the sections 1 and 2 of Fig. 32, we get the equation:—

$$p_1 + \frac{V_1^2}{64.3} = p_2 + \frac{V_2^2}{64.3} + F$$

where **F** is the friction loss from point 1 to point 2; that is, in the up-stream cone. Our practice in this series of articles has been to express **F** in terms of the velocity head, but for some reason it has been the custom, in connection with this device, to express **F** in terms of the pressure drop **H**. That is to say, **F** is written equal **bxH**, where **b** is a factor which depends on the smoothness and proportions of the meter. This method simplifies the meter formula. Further

$$V_1 = \frac{A_2 V_2}{A_1}$$

or, in terms of diameters **D** and **d** we have

$$V_1^2 = \left(\frac{p}{D}\right)^4 V_2^2$$

and the equation becomes

$$H = p_1 - p_2 = \frac{V_2^2}{64.3} - \frac{V_1^2}{64.3} + F$$

From this it is easy to write the relation

$$V_2 = C \frac{1}{\sqrt{1 - \left(\frac{d}{D}\right)^4}} \sqrt{64.3\,H}$$

The letter **c** has been written instead of $\sqrt{1-b}$. The quantity **c** is called the meter coefficient and for throat velocities exceeding 10 ft. per sec. its value is usually less than 0.99 in most meters of ordinary sizes and proportions. For the small meters **c** decreases rather rapidly for the low throat velocities and particularly those under 5 ft. per sec.

Since these articles are written for practical water works men and not for experts on metering, a discussion on the exact value of **c** for any case will not be given but those interested will find full data in the report of the Fluid Meters Committee of the American Society of Mechanical Engineers, available by applying to the Society at 29 West 39th Street, New York 18, N. Y.

For meters in pipes over two inches diameter, where the throat velocity exceeds 5 ft.

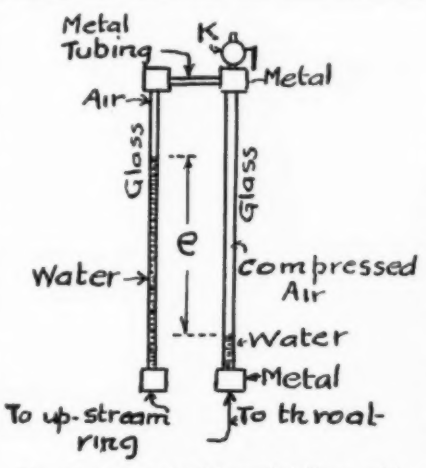


Fig. 33. Inverted Differential Gage with Metal Fittings

per sec. the value is between 0.97 and 0.99 in almost every case, the higher values belonging to the larger meters. A meter for a given sized pipe is usually selected so that the throat velocity, within its range of working, will lie between 3 and 30 ft. per sec., the extreme values being avoided as far as possible. The throat diameter is usually between one-half and one-third of the pipe diameter, but the ranges vary and the builders should be consulted on this point.

The discharge through a meter in a 30 in. pipe where the throat diameter is 15 in. and pressure differential is H = 9 ft. of water is found as follows:

$$\frac{1}{\sqrt{1-\left(\frac{d}{D}\right)_4}} = \frac{1}{\sqrt{1-\frac{1}{16}}} = 1.032$$

The quantity is usually referred to as the "velocity of approach factor" since it takes into account the approaching velocity of the water in the pipe. Putting this value in the meter formula and taking c as 0.99 (a good value for this case) gives the relation

$$V_a = 1.032 \times 0.99 \sqrt{64.3 \times 9}$$

from which the throat velocity is found to be 24.58 ft. per sec. Since the area of the 15 in. throat is 1.227 sq. ft., the discharge is 1.227 x 24.58 or 30.16 cu. ft. per sec. This corresponds to 19.54 million U. S. gallons per 24 hours.

The pressure difference p_1-p_2 or **H** can, of course, be taken by putting pressure gages at points 1 and 2 (Fig. 32) and then subtracting their readings. Such a method is not very accurate because the pressures are usually large and their difference small and any slight errors in reading the gages give a proportionally large error in **H**. As a general rule, the differential is read directly on a *differential gage* the simplest form of which is shown attached to the meter in Fig. 32. In this simple form it is a U-shaped glass tube, the two legs of which are shown connected with the pressure points by metal tubing or rubber hose.

Differential Gage

To explain the action of the differential gage, suppose that water is flowing through the meter; then the part B C D of the gage should contain a liquid that will not mix with the water, mercury being frequently used for the purpose. Extreme care must be taken to fill the connecting tubes with water from the pipe and to see that this water is free of air bubbles or foreign matter; this is accomplished by opening the pet cocks shown and allowing water to flow freely through the tubes till all air and dirt are driven from them. If either pressure is less than atmospheric, this process must be modified. On the drawing, in dotted lines, imaginary pressure gages are shown attached to the U tube at the points B and G, and a little thought will convince the reader that both of these gages will show the same pressure because there is no flow through the U tube. Obviously if one of the gages read higher than the other, the mercury in the part B C G would flow toward the point of lower pressure and there would not be equilibrium of the mercury in the differential U tube gage.

But the imaginary gage at B reads p_1 + y + e feet of water since there is nothing but water in the connecting tube, and the imaginary gage at G would indicate a pressure p_2 + y + e feet of water if all the liquid between G and the meter was water. Actually, however, the part **e** is mercury, a liquid 13.6 times as heavy as water, and, therefore the pressure produced by **e** feet of mercury would be the same as that produced by a water column 13.6 times as high.

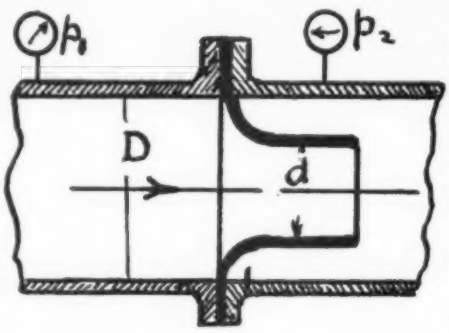


Fig. 34. Flow Nozzle

It is therefore possible to write the relation p_1 + y + e = p_2 + y + 13.6e, or H = p_1-p_2 = 13.6e-e or 12.6e. If **E** is the number of inches corresponding to **e** feet, then E = 12e and the differential

$$H = \frac{12.6\ E}{12} = 1.05\ E.$$

That is **H** is just 1.05 times the differential in inches shown in the U tube. Thus, if the U tube had been attached to the meter given in the illustration above it would have shown a difference of

$$\frac{9}{1.05} = 8.57 \text{ inches}$$

Obviously the height **y** does not affect the result and the gage may, therefore, be placed above or below the meter and at any convenient distance from it without affecting its reading. The same theory will apply to all positions, but minus signs would have to be placed before the **y** terms if the gage is *above* the meter. The reader should use the theory to prove for himself that the gage will read the same on the meter for a given discharge irrespective of whether the meter is vertical, horizontal or sloping.

Inverted Differential Gage

For very accurate work the author has used the differential gage in a somewhat different way, as illustrated in Fig. 33. The connections are made in the same way as before but mercury is not used and the gage is inverted. Air under pressure is forced in through **K** till the water in the two sides of the gage separates, as illustrated. Using the same theory as before it will appear that the differential **e** is the head **H** to be used in the formula. (Actually the weight of air in the right hand leg should be allowed for but it would produce no significant change in **H**).

Using the gage in this way on the meter mentioned would give a difference **e** of 9 ft., as compared with 8.57 in. if mercury is employed, and while this means a very much larger gage, it also means improved accuracy, since there is bound to be some error in reading **e** and the percentage error will be less with the greater difference **e**.

The differential gage is very extensively used, particularly where great accuracy is desired or where the pressure drop being measured is small. For very small pressure differences, such as are often encountered in friction and velocity head measurements, the inverted gage is very common, but some such fluid as kerosene is forced into the top of the gage instead of the compressed air already mentioned. The kerosene does not mix with the water, and differs little from it in weight per gallon, with the result that the differential **e** on the gage becomes much greater than **H**, thus magnifying the readings very much. For example, kerosene weighs about 0.82 times as much per gallon as water and the theory already explained for the mercury gage may be adapted to this case by placing 0.82 instead of the 13.6, but since the gage is inverted the signs must also be changed so that the formula will read 1-0.82 instead of 13.6-1. The net result is that with the inverted kerosene gage **H** is 0.18**e**, that is to say, an actual pressure drop of 1.8 in. would show 10 in. on the gage.

In exact experimental work, and also in practice, the principle just given is in common use and by employing various liquids of suitable weights the differential may be magnified as much as desired. The magnification will be greatest with a liquid nearly the same weight as water.

The makers of Venturi meters also supply indicating and recording instruments along with them, these devices showing directly the total, as well as the rated flow. Most of them are differential gages using mercury but the U tube is enlarged at both ends so that floats may rest on top of the mercury in both legs, these floats being connected to a mechanism above which the square root of their difference in levels is automatically extracted, and converted into gallons per day or other desired units. Details of the recording instruments may be obtained from the makers. Those who do not want to make their own differential gages may purchase them, those sold being usually part metal so that straight glass tubes may be used in them.

Flow Nozzles

In the previous article of this series it was stated that nozzles are often used as accurate flow measuring elements and an illustration of one placed on the end of a pipe was given as Fig. 31. More commonly they are inserted *in* the pipe line between two flanges as shown in Fig. 34, but if great accuracy is desired the nozzles must be of correct shape and extreme smoothness. Furthermore, the position of the pressure connections must be accurately located. Some meter builders make flow nozzles, and as they have had much experience in their design, and have determined the coefficients for them, it is best to consult them. The coefficients they use have been determined for special positions of the pressure taps, and are not correct for any other positions.

The formula for the flow nozzle is exactly the same as for the Venturi meter. In fact, the main difference between the two instruments is in the omission of the long expanding down stream cone in the case of the flow nozzle. The expanding cone of the Venturi tube has little effect on the measuring qualities but its important feature is that it does cause a reconversion of the high throat velocity energy into pressure head and, thereby, reduces the over-all loss of head in the device to a small value.

Checking Meter Indicators

The indicating device used with both of these meters may easily be checked in place by the station operator. The meter must first be examined to see that it is clean and undamaged and that the pressure openings are in good condition. After the valves in the pipes connecting the meter to the indicator are closed, the plugs in the indicator stand, close to where the connecting pipes join it, may be removed. Vertical, open topped glass tubes not less than ⅜ in. bore are connected to these plug holes (using short rubber connectors if necessary) so that the tubes are conveniently close together.

Water is then poured into the top of the glass tubes till the level in the one connected to the upstream end of the meter stands, let us say, 2 ft. above that in the glass tube

connected to the throat. This corresponds to a differential of 2 ft. on the meter, and by using the meter formula, the flow corresponding to this differential may be calculated and compared with that shown on the indicator.

It is best to carry this out for several differentials and if the dial does not indicate correctly it is usually advisable to obtain help from the meter maker. In making this check the greatest of care must be exercised in freeing all pipes and connections from air bubbles, (by no means an easy job), since just a very little air will produce erroneous readings.

SECTION XI

The Orifice Meter

To complete the work on meters for closed pipes it is essential to discuss the orifice. The orifice, which has come so largely into use during recent years, is of such a nature that it may often be put into old pipe lines with very little trouble or expense. As used in a meter, the orifice is a circular hole bored in a clean, thin, smooth plate and usually set in the pipe so that it is central with the pipe axis (the case where it is set off-center is not included in this discussion). The hole must be carefully and accurately bored so that there is no burr on its edges, but the edges must not be rounded, and for that reason the orifice is referred to as "square edged."

The plate must have a thickness not over one-thirtieth of the pipe diameter, but it is evident that it must be stiff enough so that it will not vibrate. For the large pipes it may be necessary to use a thicker plate than that specified above, but where this is done the hole must be carefully bored and the down stream edge of it bevelled away at 45 degrees till the cylindrical part left has a length not over one-thirtieth of the pipe diameter. Since wear and rust must be avoided, the plates are often made of monel metal or stainless steel. An orifice of the kind described is shown set up in a pipe in Fig. 35 and should have at least ten feet of straight pipe on the upstream side of it.

The jet passing through the orifice shows marked contraction and reaches its minimum area of about 62 per cent of that of the bored hole at a distance of approximately the orifice diameter below the face of the plate, but this distance varies with the ratio of the orifice and pipe diameters. This contracted jet is usually referred to by its Latin name of "vena contracta," the latter term often being used to denote the section of the contracted jet closest to the orifice.

The positions of the pressure connections vary, but those in the figure are a common arrangement and are designated "vena contracta taps," but sometimes flange taps are used, the holes for these being made at the pipe flanges close to the plane of the orifice. In all cases the taps are best located on the lower side of the line, and since the coefficients to be applied to the device will vary with the position of these pressure connections, care must be taken to see that the coefficients used fit the particular arrangement. The ratio d/D of the orifice to the pipe diameter usually varies from one-fifth to three-quarters, although other proportions are used.

The formula for discharge through the orifice is similar to that for the Venturi meter and for the flow nozzle, and if vena contracta taps are used, with the up stream tap far enough above the orifice (about one pipe diameter) to be clear of the disturbed part of the stream, the formula would be the same as that for a Venturi meter with its throat at section 3, although the coefficient used would not be the same in the two cases. Practical calculations, however, cannot be based on the area of the contracted jet, as there is no convenient method of measuring this area in the plant; it is really a job for a research laboratory. In actual meters the diameter d is known to within one one-hundredth of an inch, and D is also known quite closely, and these two dimensions alone are involved in the practical formula. The coefficient obtained for the meter is based on these two diameters and does not include the vena contracta.

Discharge Through the Orifice

If the reader will again review the work on the Venturi meter he will readily see that the formula for the orifice should be

$$v_2 = \frac{1}{\sqrt{1-\left(\frac{d}{D}\right)^4}} \cdot c\sqrt{64.3\,H}$$

where H is the drop in pressure $p_1 - p_2$ measured in feet of water, assuming water is the liquid flowing in the pipe. Were it not for the contraction of the jet, c would be very nearly unity as with the Venturi meter and the flow nozzle, but since in the orifice meter c takes account of the fact that the area of the jet is only about 62 per cent of that of the orifice, the orifice meter coefficient is in the neighborhood of 0.60. The meter companies have spent much time and money in getting accurate values of c and information on this point is given in the report of the Fluid Meters Committee of the Amer. Soc. of Mech. Engrs., referred to in Section X of this series and many details are also given in the Reference and Data Number of *Water and Sewage Works* for April, 1946 (p. 129). The differential H is usually measured by differential gage and recorders of the same type as is used on the Venturi meter are employed, the calibration of them being done in the way already described.

Where permanent and accurate records of discharge are to be kept, the orifice and its recorder must be carefully and properly installed and the correct coefficient must be used. It is usually best to have the installations made by a meter manufacturer. The recording instrument must be bought in any event, as it is too difficult for ordinary people to make it. It may happen, however, that measurement of the discharge is only to be made temporarily and that results within about two or three per cent are close enough. In this case a coefficient of 0.60 will serve very well on pipes 2 to 14 in. in diameter if the orifice is between one-fifth and three-quarters of the pipe diameter and vena contracta taps are used. The orifice plate must be made, as already described, and the size of the orifice should be such that the differential will not exceed a 200 in. water column, or if a mercury differential gage is used on a water line it should not indicate over 200/12.6 or 16 in. (See Section X, discussion of "Differential Gages.")

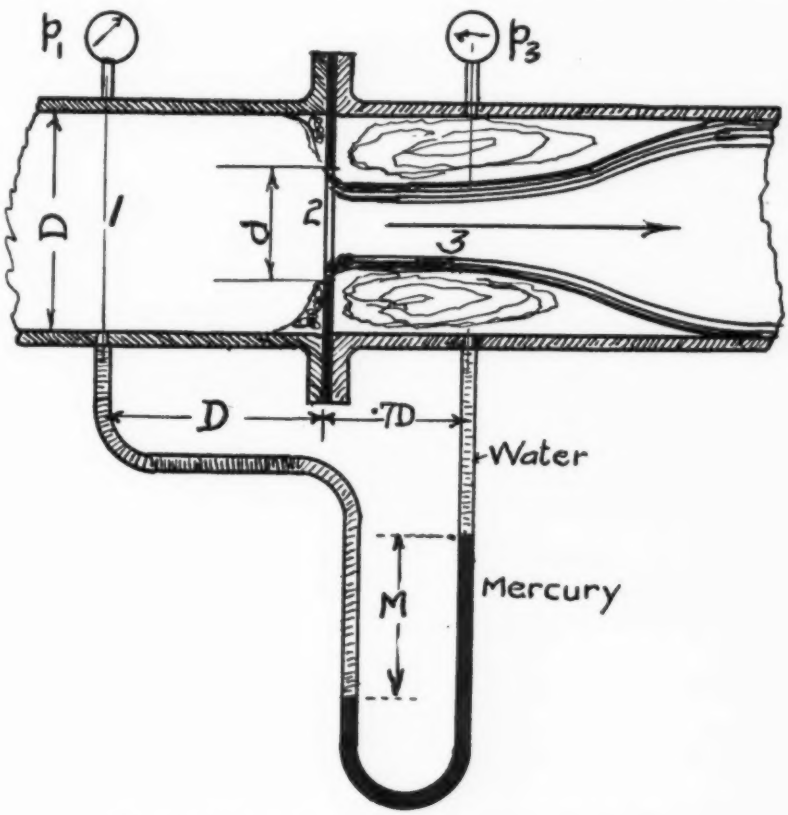


Fig. 35. Orifice in Pipe with Mercury Differential Gage

Suppose that it is desired to measure the water flowing through a 12 in. pipe, results within 2 per cent being close enough, and that it is possible to insert a plate with 6 in. orifice between flanges where there is 12 ft. of straight pipe up stream. To get the differential, ¼ in. holes should be drilled through the bottom of the pipe, one of them 12 in. (one pipe diameter) above the up stream face of the orifice and the other 0.7 x 12 or 8 in. below the same face and the ends of these holes inside the pipe wall should be examined to see that there are no roughnesses around them. Usually ¼ in. piping is used to connect the drilled holes to the gage, but this ¼ in. pipe must not

project through the inside of the pipe wall. If, after the gage has been connected up, it shows a differential of 15 in. of mercury, then the drop in pressure across the orifice is 15 x 1.05 = 15.75 ft. of water (see Section X), that is, H is 15.75 ft.

Referring to the formula given above, the velocity of approach factor

$$\frac{1}{\sqrt{1-\left(\frac{d}{D}\right)^4}}$$

will first be calculated, and since d/D is 0.5, this factor is readily found to be 1.032. The velocity through the orifice is, therefore, $v_2 = 1.032\ c\sqrt{64.3H}$, which works out to 19.7 ft. per sec. for values of c and H of 0.60 and 15.75, respectively. Looking now at Table I herewith, the discharge is seen to be 1,000 x 19.7/11.33 or 1,738 U. S. gal. per min.

The measurement is convenient to make in general and the calculation is not difficult. The results cannot be automatically recorded without a proper device, but for occasional work it is no great task for an operating engineer to read the gage at regular intervals and mark down the results.

To assist in making such calculations, as are required in the above example, Fig. 36 has been prepared and although drawn on a small scale, may be used to give accurate results. It is calculated from the formula given for the meter using c = 0.60 and an orifice diameter of one inch, for which the area is 0.00545 sq. ft., and for this case the discharge is $0.00545 \times 0.60\sqrt{64.3\ H}$ multiplied by the velocity of approach factor. But as this discharge is in cubic feet per second, it is to be multiplied by 450 to give the corresponding rate in U. S. gallons per minute. If, further, the result is desired in terms of the reading on the differential gage, suppose the latter is denoted by M inches, then since H is 1.05 times M, the formula for discharge through an orifice one inch in diameter, with a differential reading of M inches on a differential gage using mercury and connected at vena contracta taps is: G = velocity of approach factor times $450 \times 0.00545 \times 0.60\sqrt{1.05M} \times 64.3$ gallons per minute, and when this is worked out it finally gives $G = 12.1\sqrt{M}$ times the velocity of approach factor.

The curve on Fig. 36 is calculated from the formula $G = 12.1\sqrt{M}$ and hence to apply it to a given actual case one would first read from the curve the quantity corresponding to the inches of mercury shown on the differential gage, and by multiplying this quantity by the square of the diameter of the orifice in inches and by the velocity of approach factor, the discharge in gallons per minute is found. For convenience a number of common values of the velocity of approach factor are as follows:

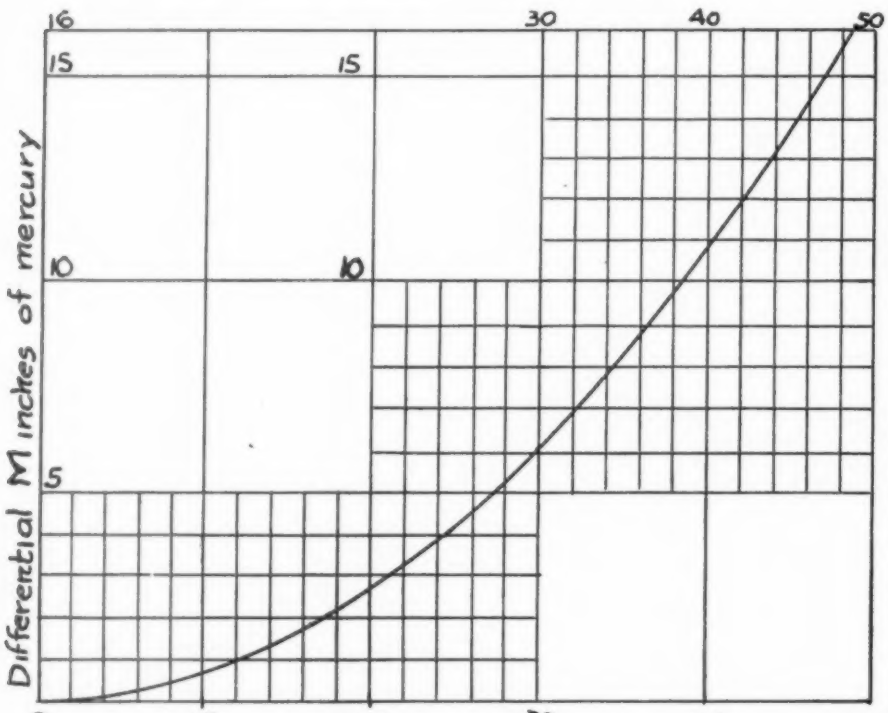


Fig. 36. Discharge Through an Orifice 1 in. in Diameter, Not Including Velocity of Approach Factor
(Multiply curve value by the square of the orifice diameter in inches and by the velocity of approach factor.)

Ratio d/D	Velocity of Approach Factor
0.2	1.001
0.3	1.004
0.4	1.013
0.5	1.032
0.6	1.072
0.7	1.147

As an illustration of the use of the curve we might apply it to the case already given, for which the quantity on the curve corresponding to 15 in. mercury differential is 46.8 and the ratio d/D is 0.5, giving a discharge of $46.8 \times 6^2 \times 1.032 = 1738$ gallons per minute. The approach factor term has little effect where d/D is less than 0.4.

Great care must be observed in the use of the orifice for it is so easily changed that a mistake may be made as to the size in use. It is common practice to make the plate with a projecting tab on which the size of the orifice is distinctly marked and this is a great help. Any dirt on the face of the orifice plate or any rounding of the edge of the orifice may cause relatively large errors, usually by giving larger discharge than the formula indicates.

Orifice in an Open Tank

The orifice is often used in the side or bottom of an open tank and makes a convenient and accurate measuring device, where water can be permitted to flow through such a tank. The constant head tank with orifices is also often used to measure and control the feeding of chemical solutions to water, sewage, etc. An arrangement of tank with orifices is shown in Fig. 37, where a square edged orifice is placed in the vertical side of the tank, with its inner face flush with the wall of the latter, so as to give a smooth plane surface all around the orifice. An orifice is also shown in the bottom of the tank. The tank must be so large that the orifice is at no point closer than about two diameters from the side walls (and bottom also in case of the vertical side wall orifice), and the horizontal area of the tank should be at least ten times that of the orifice. For accurate work the depth of water above the highest point of the orifice must be at least three times the diameter of the latter, since under this condition the velocity at the center of the orifice is, without appreciable error, the average velocity of the stream as a whole.

Where sufficient care is taken to keep the water in the tank free from eddies and whirls, and the design and location of the orifice are correct, the jet from the orifice takes on the impressive appearance of a clear glass rod of the shape shown in Fig. 37 and it has no apparent motion whatever. The vena contracta (3 in., Fig. 37) is very nearly one-half the diameter of the orifice (d) away from the face of the orifice. Its area is almost exactly 62 per cent of that of the orifice, irrespective of its size or of the depth of water above it, which means, of course, that the diameter of the contracted jet is $100\sqrt{.62}$ or 78.7 per cent of that of the orifice. Taking the datum line through the center of the orifice and section 1 (Fig. 37) several feet back, where the velocity is negligible, it is seen that the only head at (1) is the pressure head p_1, which is usually designated by the letter H in this case. In the plane of the orifice at (2) the elevation is zero and the friction loss F_2 between 1 and 2 must be small because of the low velocity in the tank.

Formula for Discharge Through the Orifice

At the vena contracta the velocity is highest because the jet is smallest at this point, and since beyond this point, the size of the jet is found to be uniform, it is reasonable to conclude that the pressure at (3) is atmospheric. Head equation then gives:

$$H = p_2 + \frac{v_2^2}{64.3} + F_2 = \frac{v_2^2}{64.3} + F_3$$

where F_3 is the loss of head in friction from (1) to (3).

The orifice was one of the devices investigated by the early workers in hydraulics, and interest in it has never ceased. It has been found relatively easy to measure the exact diameter at (3) and by catching the flow in a weighing tank on scales the flow per second is obtained and consequently the velocity v_2 through the orifice is easily determined. All the measurements show that the

coefficient of discharge $\frac{v_2}{\sqrt{64.3H}}$ varies

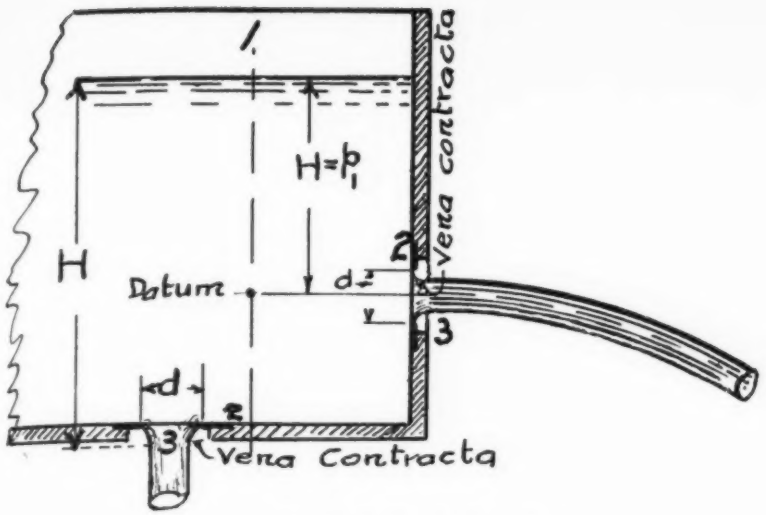


Fig. 37. Orifice In Open Tank

from 0.59 to 0.618 for round orifices from one to twelve inches diameter and for high and low values of **H**, as long as the conditions described at the beginning of this discussion are observed.

Taking a mean value of 0.61 for this coefficient and putting it in the last formula gives $v_2^2/64.3 = 0.37H$, and since it has already been stated that A_2/A_2 has a value 0.62, it follows that

$$v_2 = \frac{A_2 v_2}{A_2} = \frac{v_2}{0.62} = \frac{0.61\sqrt{64.3\,H}}{0.62} = 0.984\sqrt{64.3\,H}$$

If this value is put in the equation $H = v_2^2/64.3 + F_2$, it gives $H = 0.97H + F_2$, which means that the friction loss from section 1 in the tank to the vena contracta (3) is only 0.03H, a somewhat remarkable result when one considers the way the water is forced past the sharp edges of the orifice. The losses in contractions like this are very small.

It is a matter of interest, though of no great practical significance, that the pressure in the jet at the orifice (section 2, Fig. 37) is greater than atmospheric. This statement is evident when it is noted that F_2 must be less than F_3 and could, therefore, almost be neglected, but if its extreme upper value of $F_2 = F_3 = 0.03H$ is used then

$$H = p_2 + \frac{v_2^2}{64.3} + F_2 = p_2 + (0.61)^2 H + 0.03\,H$$

from which $p_2 = 0.60H$. For an orifice under two feet head, the pressure in the jet in the plane of the orifice is, therefore, 1.2 ft. above atmospheric.

For the orifice, the discharge, expressed in cubic feet per second (Q), is

$$Q = A_2 v_2 = A \times 0.61\sqrt{64.3\,H} \text{ or } Q = 0.0267\,d^2\sqrt{H}$$

where the diameter of the orifice is **d** inches. (The areas are all in square feet, of course.)

The above formula is a most convenient one, since it may be applied to all liquids if they are not too viscous. The round or concentric orifice is, therefore, very frequently used for measuring oils and chemical solutions as well as water. For water works problems it is most convenient to have the discharge in gallons per minute, which is easily obtained by multiplying the cu. ft. per second discharge (Q) by 450. Since the discharge (G) in U. S. gallons per minute is 450 × Q, if this is worked out, it gives $G = 12.01d^2\sqrt{H}$, where **d** is the orifice diameter in inches.

The orifice is often placed in the bottom of the tank, as is also indicated in Fig. 37, but if the head equation is written out for this case it will be found that the head **H** must be measured from the plane of the vena contracta (3), as is also indicated on the diagram, since it is at this point that the pressure first becomes atmospheric. The accuracy in this case is not quite as great as for the vertical orifice in the side wall because the **exact** position of the vena contracta is uncertain.

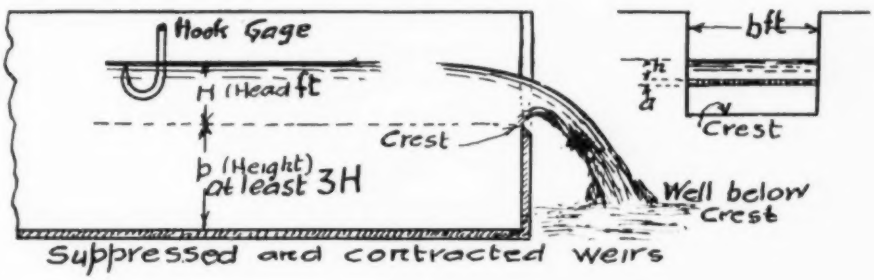
It might be well to note in passing that if the orifice is made with well rounded edges, instead of the square edges discussed in this article, there would be no contraction of the jet and the coefficient of discharge would be almost unity. (In connection with this discussion of "Orifices" the reader is also referred to Section VIII which discusses "Tank and Reservoir Discharge Openings" of various designs, including the plain circular orifice.)

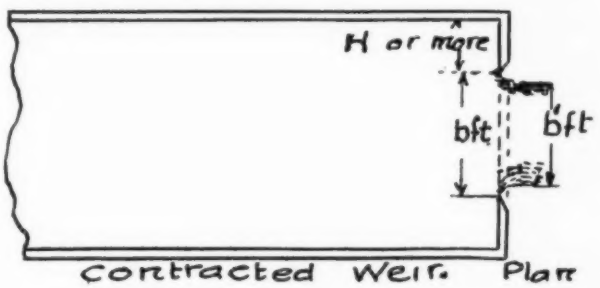
SECTION XII

Flow Measurement by Weirs*

Since the problem of measuring the discharge has already been introduced it might be well to finish the matter at the present time. Various devices in closed pipes and the orifice in an open tank have already been discussed, but the orifice is rarely used in the latter way for measuring large quantities of water, mainly because of the practical difficulties attending its use. For instance, a large orifice involving a large tank would be necessary and it would have to be deep to provide for a high head on the orifice. This makes the arrangement inconvenient and expensive, and it is difficult to avoid eddies in the tank with the large flows. For these reasons large discharges are frequently measured by *weirs*, certain forms of

*The reader is referred to two nomographs on weir discharge (rectangular and 60° and 90° V-notch) which appear on pages R-272 and R-273 in the July, 1947, number of *Water & Sewage Works*.





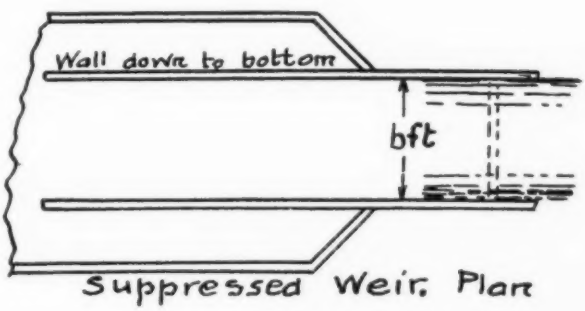


Fig. 38. Rectangular Weirs

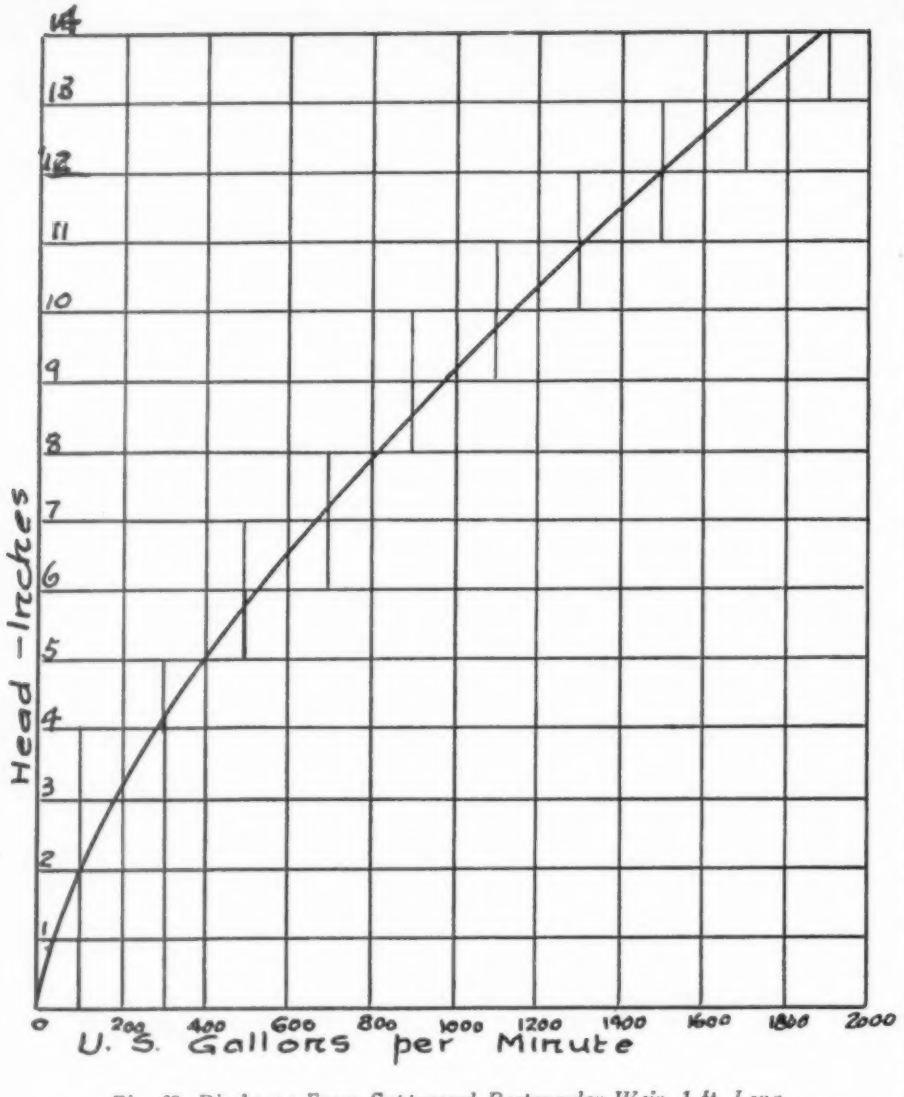


Fig. 39. Discharge From Suppressed Rectangular Weir, 1 ft. Long
(For suppressed weirs, multiply the curve value by the length of weir in feet.—For contracted weirs multiply the curve value by the weir length in feet minus one-fifth of the head in feet.)

which are often suitable for small discharges.

Rectangular Weirs

One of the most common forms of this device is the rectangular weir, shown in place in a flume in Fig. 38. In this case a dam has been placed across the flume and in it has been cut a rectangular notch, or weir, as it is called, through which the water flows, the depth of water above the horizontal edge or crest being clearly a measure of the discharge. In using the device for measuring, the edges are always made perfectly straight and smooth, with the crest perfectly level and thin, and the upstream face of the wall and weir must be very smooth with no projecting parts of any kind.

Weirs are sharp-crested, which means the same thing as square-edged in the case of the orifices already described, and the crest must be kept well above the bottom of the channel. The accuracy is very much improved if the "height" **p** is kept at least three times the depth of water on the crest, and it is better to exceed this proportion where possible. As shown on the upper diagram of Fig. 38, the stream has contractions on the lower side as well as on both ends and is referred to as a weir with end contractions, or a contracted weir; this is probably the most common arrangement. The vertical edges must be at least a distance **H** away from the walls of the tank.

Another arrangement, shown on the lower diagram of Fig. 38, has vertical, parallel walls above the weir so as to guide the water to it and thus prevent the end contractions without interfering with that at the bottom. This weir is described as one with end contractions suppressed or a suppressed weir and, naturally, the discharge over a suppressed weir will be greater than that over a contracted weir of the same length **b** and with the same depth of water **H** over the crest.

There is little to choose between the two forms, as both are equally accurate and convenient for the measurement of large flows in streams and from pipes, and they are often used to measure sewage and fluids of that kind. If errors of a few per cent are not serious, the device is easily set up, but where great accuracy is desired, much care is necessary to see that the water flowing toward the weir has no eddies in it and that the flow is evenly distributed over the entire crest; also that the flow approaches the notch in straight lines normal to the face of the weir, conditions very difficult to carry out in practice.

In all cases baffles must be arranged above the weir to remove all turbulence and to make sure that the water in the approach channel will be evenly distributed, without eddies, and with a smooth surface. Whatever arrangement is used, the water on the downstream side must be at least eight inches below the crest. Rectangular weirs have been made up to 100 ft. or more in length and sometimes have a depth exceeding six feet on the crest, but they also go down to lengths rarely under one foot, with depth of water usually over two inches over them. Their range of usefulness is obviously very great. For the shorter weirs it is good practice to limit the head **H** to less than about one-quarter of the length **b**.

It is not desirable here to enter into a discussion of the very extensive investigations that have been made on weirs, as these are of little practical value to water works men, although such studies are necessary where a high degree of accuracy is desired. The theory will, therefore, be explained very briefly by reference to Fig. 38, where a very small strip of the stream is shown hatched; its height is **a** ft., which is very small, its length **b** ft., and its area is **ab** sq. ft., if there is no contraction at the ends, as in the suppressed weir; but if there is end contraction, the effective length is less than **b** and the effective area must be corrected accordingly. In introducing the matter, the suppressed weir is being considered. From previous discussion on velocity and velocity head, the velocity through the little strip is $\sqrt{64.3\ h}$, where **h** is the depth of water on the strip, measured in feet. The volume of water flowing through this strip is proportional to $ab\sqrt{64.3\ h}$, and if the whole depth of water on the crest is divided into similar strips, the total discharge is the sum of the separate discharges through the individual strips.

But all the strips have the same length **b** and the sum of all the small heights **a** is **H**, the total depth of water on the crest, and if all the small discharges through the separate strips are added together to give the total discharge, the latter will be expressed in a formula like $Hb\sqrt{64.3\ H}$ multiplied by a number which will take account of friction and end and bottom contraction as it occurs. It must, however, be observed that the velocity through the various strips varies greatly, that of the top strips being much lower than that of the strips near the crest, and that the formula just quoted gives the discharge as proportional to the area of the water **bH** at the weir, multiplied by the velocity $\sqrt{64.3\ H}$ at the crest, whereas it should be multiplied by the average velocity. It is not desirable here to give the mathematical discussion which shows that the average velocity is two-thirds that at the crest.

Friction and contraction have an effect similar to that in the orifice and reduce the discharge to about 62.3 per cent of what it would be if these were absent. (The coefficient of discharge for the orifice has already been given as 0.61, which corresponds with the 0.623 for the weir where a slightly higher result would naturally be expected.) The discharge for the weir would then be given by

$$Q = \frac{2}{3} \times 0.623bH\sqrt{64.3\ H} \text{ or}$$

$Q = 3.33\ bH\sqrt{H}$ in cubic feet per second. Both **b** and **H** are in feet. Literally thousands of experiments have been made on weirs, one of the first extensive series being made by Francis at Lowell, Mass., nearly one hundred years ago, where he

had a way of catching the flow in a large reservoir and measuring it directly. He compared the measured discharge with that calculated from the above formula and found a remarkably close agreement for suppressed weirs provided the bottom of the approach channel was at least three times **H** ft. below the crest, but for smaller distances (heights) the formula had to be modified. Francis also found that the same formula would apply to contracted weirs provided the length b was replaced by $b-\frac{H}{5}$ and these formulas $Q=3.33\,bH\sqrt{H}$ for suppressed weirs and

$$Q = 3.33\left(b-\frac{H}{5}\right)H\sqrt{H}$$

for contracted weirs are called "Francis formulas." The discharge is in cubic feet per second and both **b** and **H** are to be in feet.

The writer is quite well aware of the defects in these formulas and that they would not be used in such accurate work as turbine and pump testing, but such highly precise work must be done by an expert in any event. The formulas have been largely used for ordinary work where an error of perhaps two per cent is not serious; they are very simple and easy to work out, a fact which gives them great merit. For a weir 10 ft. long with a depth of 18 in. on the crest (**H**=1.5) the discharge would be $3.33\times10\times1.5\sqrt{1.5}$ or 61.2 cu. ft. per sec. for a suppressed weir and $3.33(10-0.3)\times1.5\sqrt{1.5}$ or 59.3 cu. ft. per sec. for the contracted weir. These results are close if the crest is at least 4.5 ft. above the bottom of the channel and there are no serious eddies upstream.

As a check on calculating the discharge by these formulas, the curve shown on Fig. 39 has been prepared. It gives the discharge over a suppressed weir one foot long for different heads in inches, the discharge being in U. S. gallons per minute. The crest is assumed to be at least a distance of three times the head above the bottom of the tank or flume. To get the discharge for any other length of weir it is only necessary to multiply the value taken from the curve for the given head by the length of the weir in feet if it is a suppressed one, or by the length of the weir in feet less one-fifth of the head in feet if the weir has end contractions. As it is hard to read the curve with great accuracy, it will generally be advisable to work out the discharge by the formula as well.

The head **H** must be measured in feet at a distance of at least three feet upstream from the weir, so as to be in the level part of the water surface, and it is very frequently taken by a hook gage of the form shown in Fig. 40. This type of gage is used because it is easy to see when the point of the hook breaks through the water surface. If the latter is perfectly smooth, this break through can easily be detected for a movement of the hook of less than one one-hundredth of an inch so that very accurate setting of it is possible. The movable rod to which the brass hook is attached is equipped with an adjusting screw which facilitates the matter and the vernier enables very close readings to be made.

The zero reading on the rod may be found by means of a level by which the hook may be set level with the crest, but a more accurate and easy way is to fill the space above the weir with water till it is just level with the crest and then set the hook to suit this level and take the

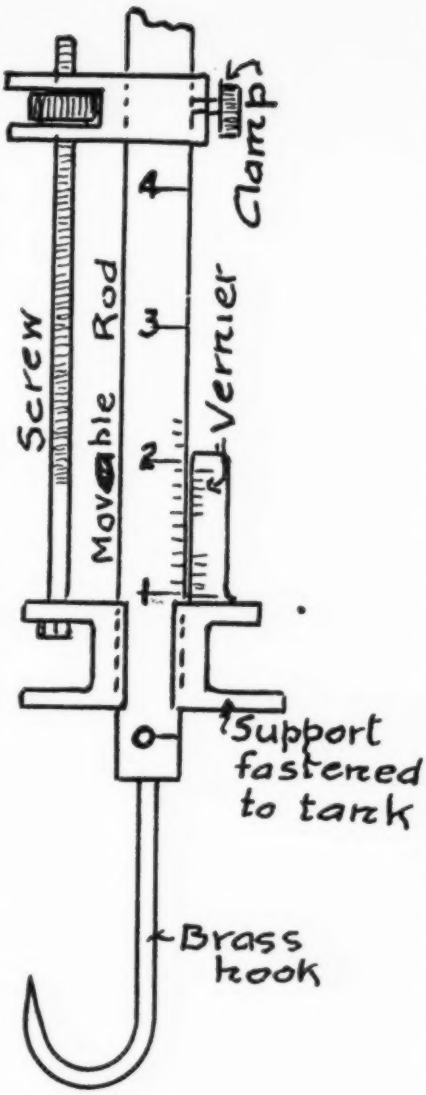


Fig. 40. Hook Gage

reading. Great care is necessary in fixing the water level because the surface tension of the water enables it to stand slightly above the crest without running over. Sometimes it is necessary to surround the hook with a stilling box placed in the water so that the top of it comes above the surface, and made with a half inch hole in the bottom, so that oscillations will be damped out, but the water in the box will be level with the average water surface.

V-Notch Weirs

Rectangular weirs are rarely less than one foot long and are not, therefore, used for measuring very small flows. Where an orifice is not convenient, the V notch is often employed and is suitable for discharges of from around 10 to more than 3500 gallons per minute, although for the upper limit other measuring devices are more common. This weir is shown in Fig. 41 and is just a notch of triangular shape cut in the vertical wall of a dam or tank. It is sharp edged as in the case of the rectangular weir and the angle at the vertex varies from 90 degrees in the right angled V notch to quite small angles where the flow is small. The notch may be cut in a smooth brass plate, but boiler plate or cast iron is often used. Since the latter two materials are not so smooth as brass, they tend to decrease the contraction of the flowing jet and usually give a greater discharge for the same head than the brass one does. Whatever metal is used, the face of the metal must be kept free of grease, paint, dirt or rust, particularly close around the edges of the notch.

As in the case of rectangular weirs, it is only possible to give an outline of the theory applying to the V-notch weir, and a reference to the drawing showing a longitudinal section and a front view will indicate the terms used. The small horizontal hatched strip has the length **b**, the small height **a**, and is at depth **h** below the water surface. The discharge through this small strip is $ab\sqrt{64.3\,h}$ multiplied by a factor to correct for end contractions and for friction. But the length **b** of each horizontal strip is proportional to **h** and consequently the addition of the discharges of all these strips making up the entire area above the crest will be proportional to $H^2\sqrt{64.3\,H}$, as is easily shown by mathematics. The formula used for the notch is $Q=cH^2\sqrt{H}$, where **c** is a coefficient taking account of friction, contraction and also the number $\sqrt{64.3}$. For the right-angled V notch **c** varies from 2.55 for a head of 0.25 ft. to 2.49 for 0.85 ft. head where the notch is in clean brass plate, but if a surface of about the same smoothness as clean, rust-free boiler plate is used, then **c** varies from 2.63 for the lower head to 2.48 for 1.8 ft. head, according to King's Handbook of Hydraulics (McGraw-Hill Book Co.). An approximate average value of 2.5 would be from 2 to 5 per cent low on the low heads and very close on the higher ones. The curve in Fig. 42 will be found useful and is for a weir cut in a plate of similar smoothness to boiler plate, the notch being right-angled.

The vertex of the notch must be at least three times the head away from the sides and bottom, and the same precautions must be taken as with the rectangular weir in damping out eddies, etc. For the 90-degree notch the breadth **B** is equal **2H**, but for measuring small flows such

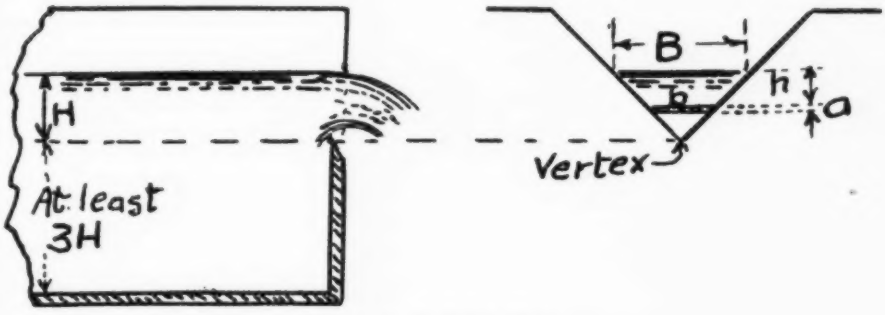


Fig. 41. V-Notch Weir

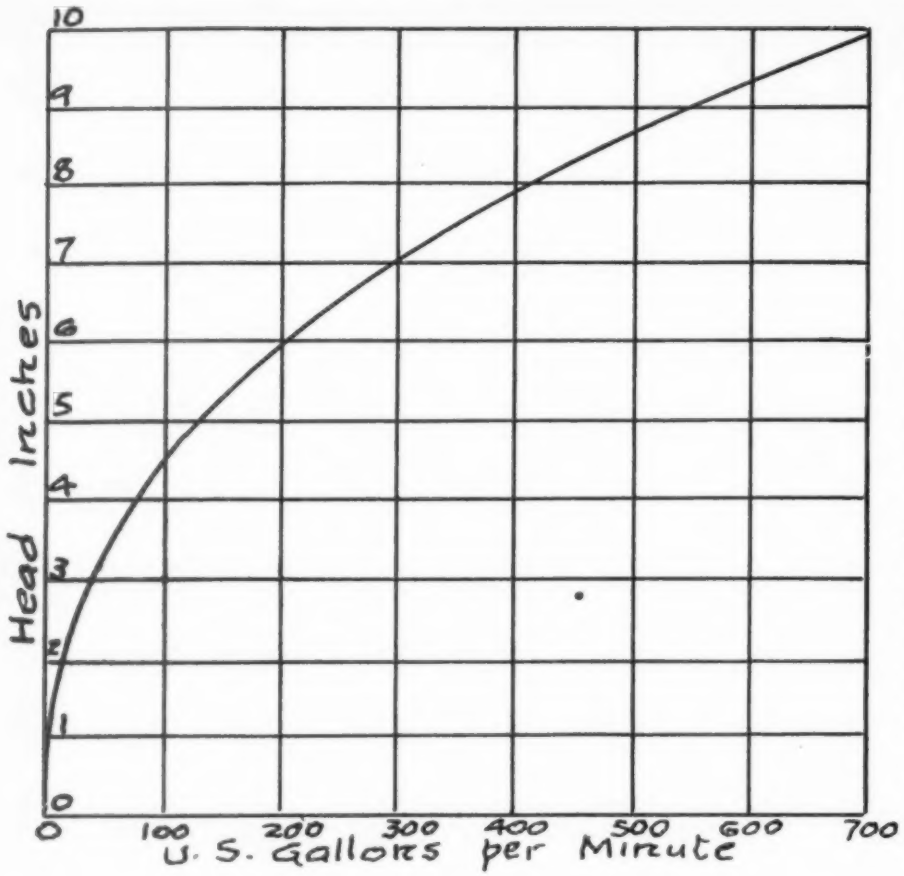


Fig. 42. Discharge Curve for Right-Angled V-Notch Weir

as feed water and chemical solutions, **B** is often made much smaller with the angle at the vertex less than 90 degrees. While it is possible to say, *very roughly*, that the discharge decreases in proportion to **B**/**H**, and that if **B** is equal to **H**, the discharge will be half that for the 90 degree notch under the same head, yet it is essential that such weirs be specially calibrated, and this is always done by the companies supplying them. This weir has many applications.

Selection of the Measuring Method

Generally one has an approximate idea of the flow to be measured. If it is small, the orifice or the V notch with small angle is most convenient, but if it is in the neighborhood of 700 gallons per minute, then a 90 degree V notch is available with a head a little under 0.83 ft. (10 in.) or a rectangular weir could be used. In the latter case it would not be desirable to use a weir one foot long with a head of nearly 6 in. because, in general, the head should not be over one quarter of the length; it would be better to use a two-foot length with a head around 4.6 in. or a three-foot length with about 3.5 in. head. Good judgment is essential. The use of the stilling box is recommended for accuracy in getting **H**.

SECTION XIII

Flow in Streams, Drains, and Open Channels

So far attention has been confined to what are called "closed channels," such as pipes, in which the flowing liquid completely filled the conductor carrying it and where the fluid could be under any desired pressure. There are a great many cases, however, where the surface of the water is at atmospheric pressure, a condition common to sewers, drains, ditches, rivers, etc., and these cases are included under the head of "open channels."

The flow in them may be discussed under the same general laws as apply to closed channels, or pipes running entirely full, but there are certain differences that must be recognized, as, for example, the fact that the closed pipe may run over hills and down valleys following in general the contour of the ground, but it is well known that open channels, such as sewers and rivers must always slope downward in the direction of flow. This is the reason for the elaborate and beautiful viaducts constructed to supply Rome with water, and demands the location of an open channel with much greater care than is necessary in the laying of pipes running under pressure.

In addition to this difference the velocity in a closed pipe full of water is proportional to the discharge and any change in the latter produces a corresponding change in the velocity. This statement is not generally true of the open channel for the depth of the water, and consequently the cross-sectional area of it, may, and usually does, vary with the discharge and it is quite conceivable that an increase in discharge may correspond with a decrease in velocity, because the area of the water may increase more quickly than the discharge. While these differences are at once evident, and others will appear as the article develops, yet the laws already laid down enable problems in open channels to be dealt with satisfactorily.

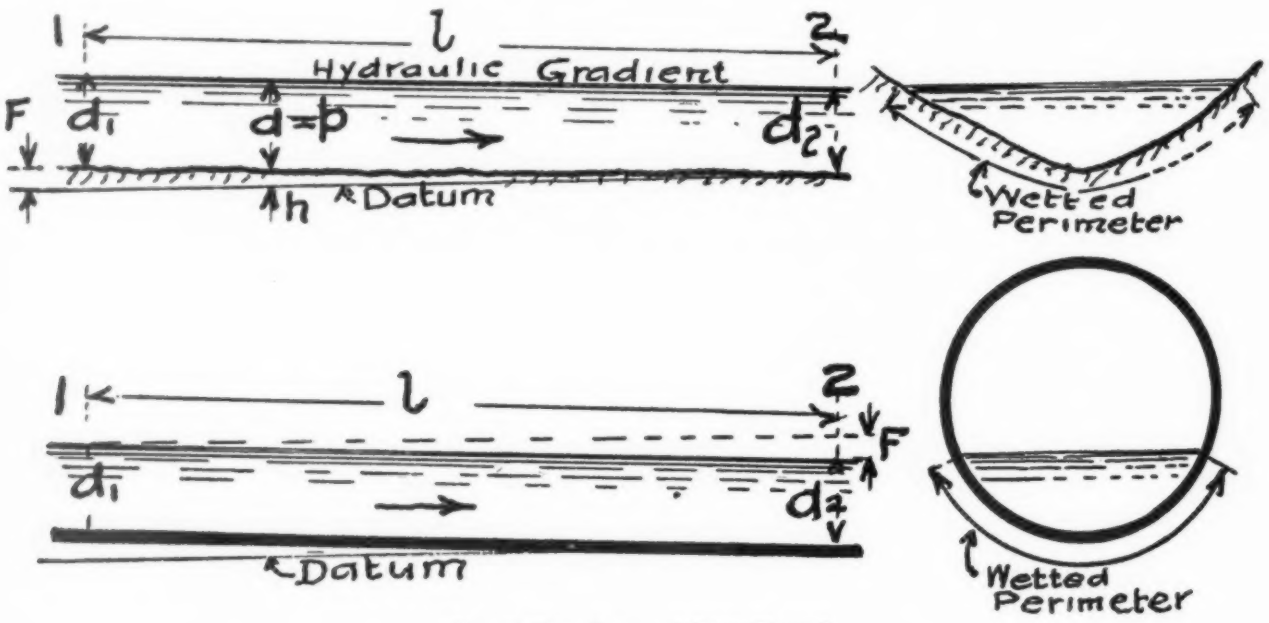


Fig. 43. Two Forms of Open Channel.

Flow in Train

The drawing on Fig. 43 shows longitudinal and cross sections of a drain pipe, or sewer, or ditch. The slope of the bed is always small, such as six inches per mile in many rivers, one foot per mile in some sewers, while one foot per mile has been used in large power canals; this is done to keep the velocity of the water low enough to prevent destruction of the bottom and sides of the canal. The two sections, 1 and 2, are 1 ft. apart and the depths at the sections are d_1 and d_2 respectively. In many practical problems both the shape and area change between the two sections; consequently there is a change in velocity from one to the other.

While hydraulic problems dealing with such cases usually offer little difficulty, yet they involve a more intricate treatment than it seems advisable to bring into this series. The study here will, therefore, be confined to the simplest case, namely that where the cross-sectional area of the water and the shape and dimensions of the channel remain unchanged throughout its entire length, and where the slope of the bed is also uniform. This is the case commonly treated in general hydraulic studies and is the condition assumed in design; it is described as *flow in train* and obviously the water surface is parallel with the bed of the channel.

The datum will be assumed through the bed at section 2, as indicated, and since the depth of water at any point corresponds to the pressure **p** on the bed at that point it is evident that, if elevation heads are measured up to the bed, then the hydraulic gradient coincides with the water surface and the resistance line is parallel with the surface and $\frac{v^2}{64.3}$ ft. above it.

Inasmuch as the velocity in open channels is usually low, the velocity head is generally small, although it may be as large as one foot or more. As all water cross-sections are assumed to be equal all velocity heads will be the same, and, further, the sum of the elevation head and the pressure head will always be the distance from the datum to the water surface and it follows that the friction head **F** ft. lost between sections 1 and 2 is simply the drop in the water surface between these two points. The slope of the hydraulic gradient $\frac{F}{l}$ is therefore also the slope of the water surface and of the bed. This slope $\frac{F}{l}$ is usually designated by **s**.

In the fifth section considerable attention was given to the method of calculating **F** in case of a closed, round pipe, and that section should be read again in connection with the present one, for the same basic laws apply to the open channel as to the former case. The law used for the pipe was

$$F = f \frac{l}{d} \times \frac{v^2}{64.3}$$

where f was a term having mainly to do with the roughness of the inside wall of the pipe, although it was stated in the article that f also varied to a certain extent with the velocity and the diameter. The diameter **d** entered in because it was a measure of the mean distance of the particles of water from the pipe walls, which are primarily the cause of the friction loss.

Formula To Be Used

If the same type of formula is to be used for open channels what dimension in the open channel will correspond with **d** in the pipe? Evidently it must be a dimension proportional to the mean distance of the particles of water from the bed, and this dimension has been arrived at in a somewhat curious way, perhaps best illustrated (although not too satisfactorily) by assuming the water flowing in a rectangular trough.

Suppose this trough is 6 ft. wide with a depth of water of 2 ft., so that the cross-sectional area of the water is 12 sq. ft. and, of course, no drop of water can be more than 2 ft. from the bed. The *wetted perimeter* of this cross section, that is the length of the perimeter in contact with the water is $2 + 6 + 2 = 10$ ft. and dividing this length into the area gives 1.2 ft. If, instead of these proportions, the width were 12 ft. and the depth 1 ft. the area would be the same as before, but the wetted perimeter would now be 14 ft. and by dividing this into the area, as before, gives 0.857 ft., which is smaller than in the former case and also the average particle is closer to the bottom than before, whereas a channel 8 ft. x 1.5 ft. would still have the same area but a perimeter of 11 ft. and the quotient $\frac{12}{11}$ is 1.09 ft. which is between the other two, as is also the maximum distance of any drop of water from the bed.

Very many experiments and studies have shown that this quotient is in every case in practice, a measure of the effect on its flow of the distance of the average drop of water from the wall. In the above example, the curious case arises that a 6 ft. x 2 ft. trough has the same perimeter as a 4 ft. x 3 ft. one and yet the maximum distance of the drops from the walls is not the same in the two cases, apparently. This in no way affects the accuracy of the statement made but suggests that in trying to simplify the matter the author has not made a comprehensive enough explanation. Actual results fully justify the statements.

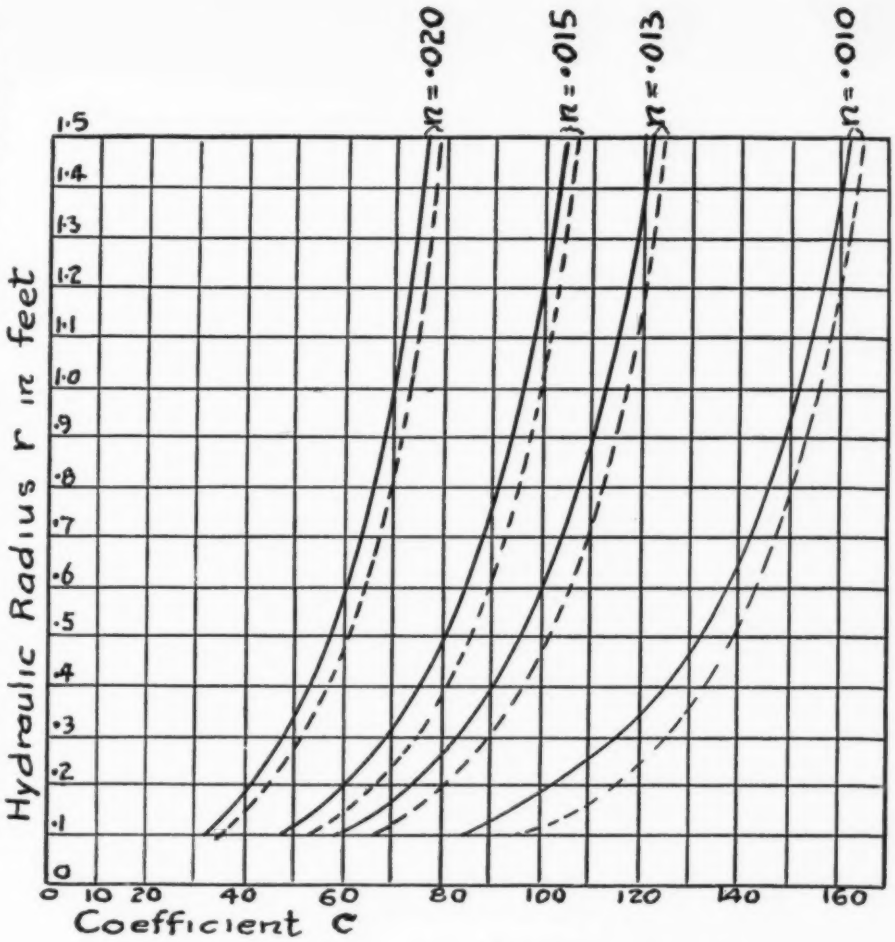


Fig. 44. Values of c in Chezy Formula, $V = C\sqrt{rs}$.

Hydraulic Radius

The quotient, area divided by wetted perimeter is called the *hydraulic radius*, or hydraulic mean depth, and is denoted by **r**; it performs exactly the same function in the formula for open channels as **d** does in the closed pipe formula. It is also true that **r** is proportional to the size of the canal, as is easily seen by a few trials; for example, in the 6 ft. x 2 ft. channel above **r** was 1.2 ft. while in a 12 ft. x 4 ft. channel it would be 2.4 ft.

With these explanations it is possible to write the relation

$$F = f_1 \frac{l}{r} \times \frac{v^2}{64.3}$$

for the open channel, and this is more commonly written in the form $v = \sqrt{\frac{64.3}{f_1}} \sqrt{r \frac{F}{l}}$ But $\frac{F}{l}$ has already been designated

by s and is the slope of the bed, which is the same as that of the water surface, while f_1 like f in the pipe formula, depends very largely on the condition of roughness of the bed and walls. Usually $\sqrt{\frac{64.3}{f_1}}$ is denoted by the coefficient c which depends almost entirely on the roughness of the bed and walls, although like f it also varies with r and v as well. But since v varies with s it is more common to say that c varies with r and s as well as with the roughness.

Chezy Formula

Putting these values in the formula gives it the form $v = c\sqrt{rs}$, which is known as the Chezy formula, from its discoverer, and it is used almost universally for flow in open channels, and quite frequently for closed pipes as well. The quantity s always denotes the slope of the hydraulic gradient and for open channels operating under the conditions named it is also the slope of the bed and also of the water surface but for closed pipes flowing full the slope of the pipe line may have little connection with the slope of the hydraulic gradient.

In fact while the hydraulic gradient slopes downward in the direction of flow it is always desirable to have the closed pipe line slope upward in that direction so that any air freed from the water will be carried along with it and not collect in the pipe. The values of the coefficient c vary greatly because of the great variation in size, shape and roughness in open channels which include glazed tile sewers, brick sewers, wooden troughs, concrete lined ditches and drains, ditches in earth where the bed is sometimes smooth and at other times rough and stony and often with grass and weeds growing in them.

The water works man is not much interested in the elaborate formulas that have been devised for giving a value of c to suit each case but he should be reminded that one of the most commonly used is that of Kutter, who included in his formula a term n to denote the degree of roughness, and this term n is in such common use that engineers with some experience indicate the condition of the bed by giving the value of n for it. If, for instance, they were told that a certain channel had a value of n = 0.015 they would know that it had a bed of about the same smoothness as vitrified sewer pipe.

Kutter's 'n' for Roughness

A few values of this coefficient of roughness n are as follows: Neat cement finish 0.010, brass pipe 0.011, coated iron pipe 0.013, unplaned board lining 0.014, galvanized iron pipe and vitrified sewer pipe 0.015, and for natural earth it varies from 0.023 when clean and straight to between 0.03 and 0.04 where there are weeds and stones. Since it has been found that the

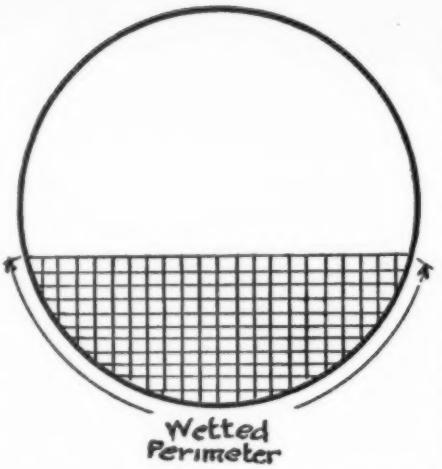


Fig. 45. Method of Finding Hydraulic Radius r.

value of c varies almost exactly as $\frac{1}{n}$ in a channel, it is most desirable to keep the walls and bed smooth, for if when the channel was new and clean n was 0.023 and it was later to have a growth of weeds which increased the value of n to 0.034 then the capacity of the channel would only be approximately 23/34 what it was at first for the same depth of water. The effect of roughness cannot be over emphasized.

For convenience, values of c are plotted on the curves on Fig. 44, which cover the usual cases met with in water works practice, but very complete tables of values will be found in King's *Handbook of Hydraulics* (McGraw Hill Book Co.) and also in *Hydraulic Tables* of the Corps of Engineers, War Department, and sold by the Superintendent of Documents, Washington, D.C. For a good, smooth concrete trough 4 ft. wide and carrying water 1 ft. deep, the bottom slope being 1 ft. per 1000 ft., or s = 0.001, n should be about 0.014 and the hydraulic radius would be r = 4/6 or 0.67 ft. for which values the curve gives a coefficient c of 98. The velocity in the trough would be

$$v = 98\sqrt{0.67 \times 0.001}$$ or

2.54 ft. per second with a corresponding discharge of 4 x 2.54 or 10.2 cu. ft. per sec., which is equal to 4590 U. S. gallons per minute. If this concrete became so dirty that the value of n rose to 0.20 then c would decrease to 65, which is nearly 14/20 of 98, and the discharge would drop to 3045 U. S. gallons per minute.

Some care would need to be exercised in the design of the above trough to see that a gradually tapering approach channel was made, which would very gradually speed up the water and thus avoid large entrance losses. If the trough was 500 ft. long then the friction loss in it would be 500s or 0.5 ft., and if the trough connected two reservoirs the difference in levels between them would be 0.5 ft., plus the entry and exit losses. These latter together might be nearly twice the velocity head or 2 x 0.1 ft. which would make a difference in levels between the reservoirs of 0.7 ft., that is 8.4 in. The velocity in this trough is quite satisfactory, but could not be used in an unlined channel, as it would probably wash out the bed.

Discharge of Round Channel

If a 2 ft. diameter round concrete pipe had been used instead of the rectangular trough, and the pipe was lined with neat cement it would have a value of 0.010 for n, and if this pipe was laid below the hydraulic gradient, so as to be always full, the water area would be 3.14 sq. ft. and the wetted perimeter 6.28 ft. giving a hydraulic radius of 0.50 ft. If the reservoirs were to have the same difference in levels as before, and the same entry and exit losses are assumed, the loss of head in friction would be 0.5 ft. as before, so that the slope of the hydraulic gradient would still be 0.001 (although the pipe itself would probably slope upward). From Fig. 44 the estimated value of c is 135 and if these values are placed in the Chezy formula the velocity is found to be 3.02 ft. per sec. and the discharge 3.14 x 3.02 or 9.48 cu. ft. per sec.

Had the 2 ft. pipe been raised till its centre line was on the hydraulic gradient it would run half full and would have the same hydraulic radius 0.5 ft. as before, so that it would have the same c and the same velocity of 3.02 ft. per sec., but its discharge would be cut in half because the water area is reduced in this proportion. For this case the pipe would need to be carefully laid to grade throughout.

Lined and Unlined Channels

Where channels are lined with a material that water cannot damage, quite high velocities are permissible, and these may even exceed 8 ft. per sec., but particular care must be taken to avoid high entry and exit losses, as these may easily exceed the friction loss.

Where the channel carries an unclean liquid, such as sewage, the velocity should be high enough to prevent the solids from falling to the bottom or sticking to the sides, for this would raise n and lower the capacity. For sewage, 2 ft. per sec. is generally high enough to prevent trouble but for other fluids it may be necessary to determine this value by experiment. On the other hand unlined channels must not have too high a velocity or the bed will be washed out. Fine sand is moved by a velocity of 1.5-2 ft. per sec., fine gravel by 2.5-4 ft. per sec. but firm loam will stand 2.5-3.5 ft. per sec. These velocities are for clean water, but if there is sand or corrosive material in it these must be placed somewhat lower.

The Hydraulic Radius 'r'

For channels of simple and regular shape there is no trouble in getting the value of r and in circular channels flowing full or half full the hydraulic radius is always one-quarter of the diameter, but for other depths and for irregular channels it is not so easy to get. The cross section of the water, however, can always be plotted accurately either full size or to scale, and if a planimeter is handy the area may readily be measured. Usually such is not the case and then the area may be divided up into small squares, as shown in Fig. 45, and the number of these can be counted which will give the area in square feet. The only way to get the wetted perimeter is to set a pair of com-

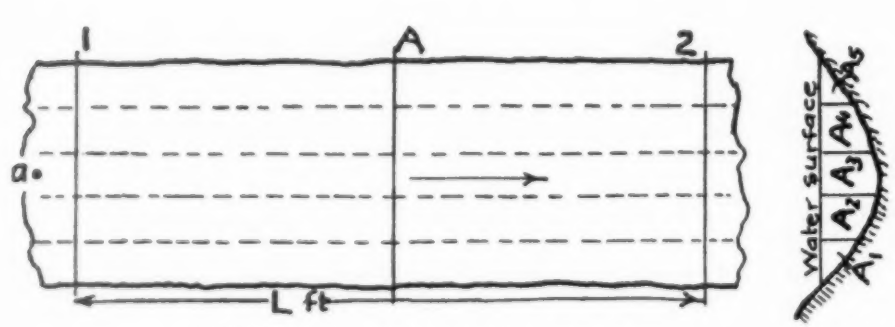


Fig. 46. Plan and Section of an Open Stream

passes an even distance, such as one inch apart and then "step off" the perimeter from the drawing. This method will give very good results if the squares are small enough and the distance apart of the compass points is not too large.

It must be remembered that this treatment of open channel flow applies only to the case where there is **flow in train,** the water surface being parallel to the bed and all water cross sections are of the same shape and size. It must not be applied to other cases.

SECTION XIV

Flow in Streams

In the previous section flow in open channels was discussed and the formulas used in connection with such channels were explained. The methods described there were those very largely used for the design of such channels but they are not employed much for finding the discharge of existing streams. The Chezy formula would be accurate for such a purpose if it were possible to get correct values of **n, r** and **s** but it is particularly difficult to select a good value of the coefficient of roughness, and frequently it is by no means easy to get a good average value of **r,** because, in many streams, the shape of the cross section varies considerably. Finally since the slope is usually small considerable proportional errors in the measurement of it are possible.

Perhaps the best reason of all for not using a formula is that there are convenient direct methods of measuring the discharge and some of these methods will be briefly described. Where the stream is a small one the simplest method of all is to set up a rectangular or a V-notch weir in the way described in connection with them in Section XII. For the larger streams such a method is generally expensive and often impracticable, but with the smaller channels a temporary dam with a weir in it is conveniently and cheaply made and a complete record of the stream is easily kept by taking readings on the head as frequently as desired. The construction of the dam and weir will always raise the water level above them and this sometimes causes objectionable flooding.

A plan and an average cross section of a stream are shown on Fig. 46 and these must be carefully plotted to scale for any case to be examined; on this same drawing two cross sections a considerable distance (usually at least 100 ft.) apart are selected. The part of the stream between these must be as straight, uniform and clean as possible and with the water flowing in straight lines down it. The drawing of the middle (mean) cross section A is next divided into a series of vertical strips of equal width, the number of these depending upon the judgment of the individual and the accuracy desired, but there should not, in general, be less than five of them. The area of each of these strips is then determined by any of the usual methods such as dividing the area up into a number of small squares of equal size and counting the squares. In the drawing there are five such strips, of areas A_1 to A_5 square feet, and the sum of these is, of course, equal to the total area of the section, which is A square feet. Having completed this part of the work the next thing is to try to determine the mean velocity of the water in each of the strips and if none of the expensive equipment, such as a current meter, is available the common practice is to use floats often made of rods

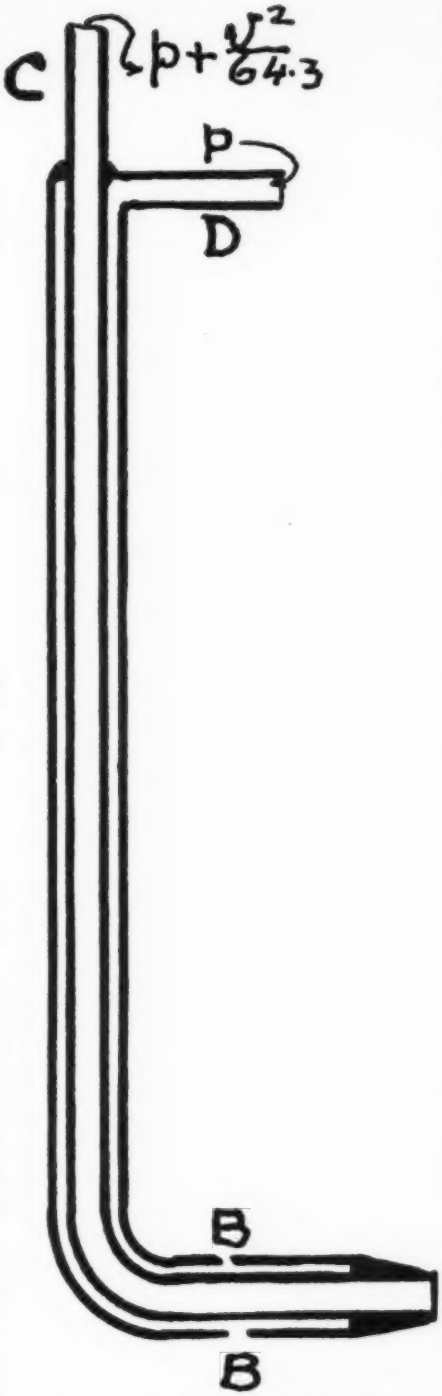


Fig. 47. Simple Form of Pitot Tube

(rod floats) and at other times of light boards which will float on the surface of the water.

Rod Floats

Where the stream is deep these are the most reliable type of all. They are made of light, dry wood with a weight at one end to make them float nearly vertically, but sometimes light tin pipes are used, closed at one end and loaded with stones or sand to bring them down to the desired depth. Bottles are often used in the shallower streams, weighted in the same way as the tin pipes are. On the plan (Fig. 46) dotted lines have been drawn corresponding with the divisions in the cross section, and with these as a guide a float is set in the stream at such a point as a and loaded so as to float as near to the bottom as possible without catching. The distance **L** ft. between the two cross sections having previously been measured, the time **t** sec. taken for the rod to float from the upper to the lower section is observed and the mean velocity of the float is readily calculated by dividing **L** by **t.**

In general the velocity of the float rod differs from the mean velocity of the water for several reasons. In the first place the rods cannot float very close to the bed because the latter is usually rough and uneven, and the rod must be kept high enough so that it does not catch at the shallowest point. Since the velocity near the bed is obviously lower than that higher up the float has a tendency to travel about five to ten per cent faster than the mean water velocity. Then again air resistance affects it and it must not project much above the surface, and finally it has a tendency to wander from one strip to another on its downward path. The float, however, gives very good satisfaction under favorable conditions and is very extensively used where better methods are not available. The mean velocity in the strip is quite close to ninety-five per cent of that of the float rod usually.

Surface Floats

These are commonly made of pieces of plank light enough to float on the surface but they should project as little above it as possible because they may be seriously affected by winds or air movement. They are used in exactly the same way as rod floats but are not as good as the latter because they are affected only by the surface currents and do not attempt to share the average velocity in the strip. As a rough average value it is fair to assume that in the ordinary stream the mean velocity in the vertical strip is not far from ten per cent above that shown by the surface float. At best they are not very reliable.

For any of the float methods the total discharge of the stream is, of course, the sum of the discharges in the several strips, and if the areas of the strips are denoted by A_1, A_2, A_3 etc. and the mean velocities in these strips are v_1, v_2, v_3 etc., respectively, then the total discharge is

$$Q = A_1v_1 + A_2v_2 + A_3v_3$$

etc. in cubic feet per second. The measurement of the discharge of a stream, or stream gaging as it is called, is a tedious and long process, requiring good judgment on the part of the observer, and it is necessary to make sure that the rate of flow is not varying while the measurements are being made. At some selected point a stake should be driven down into the bed till the top of it is flush with the water surface so that any variation in depth may be observed. Should such variation occur the work must not proceed.

The Pitot Tube

In Section II reference was made to the Pitot tube and its use in measuring the discharge of fire streams, but as its application has been quite extensive, particularly in water works surveys, it will be well to describe it more in detail. Its practical application covers a period of over 200 years.

Two forms of the instrument were shown in Figs. 13 and 14 but these were only suitable for streams at atmospheric pressure and in which the pressure in the tube indicates the velocity head only. If either of these tubes was inserted into a pipe line under pressure, such as a water

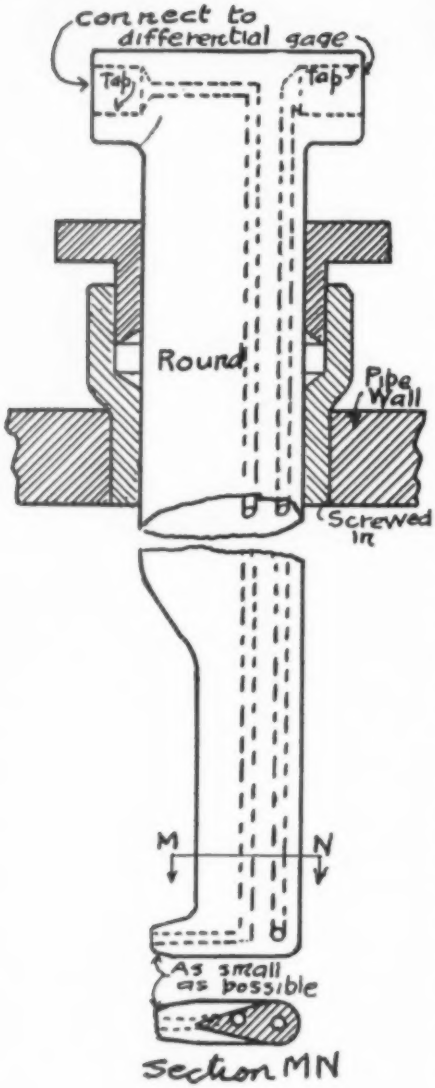


Fig. 48. Pitot Tube for Large Pressure Pipes

works main, then the gage attached to it would read the sum of the pressure and velocity heads as was pointed out in Section II. To get the velocity head alone some method would have to be taken of subtracting the pressure head, and the above mentioned section has pointed out why this cannot be done with accuracy by subtracting the reading of a pressure gage attached to the wall of the pipe from the reading of the gage on the tube of Figs. 13 and 14. A special form of tube is always used on pipes under pressure.

One of the simplest of these forms is shown in Fig. 47 and is made of two brass tubes bent as shown, the tubing being of as small a size as will be strong enough to withstand pressure and shock. There is sufficient clearance between the inner and outer tubes to allow free passage of water and the entrance of the tube is usually fairly sharp. From what has been said the inner tube is under pressure due to the sum of the pressure and velocity heads while the space between the tubes is affected only by the pressure in the pipe, this being transmitted through the holes B. Consequently if a differential gage (Section X) is attached between the points C and D of the Pitot tube this gage will indicate the velocity head directly.

The Pitot tube may be used to measure discharge in a pipe without interfering in any way with its operation, for it is possible to design a tube that can be inserted through a three-quarter inch corporation stop cock. One such design is shown in Fig. 48. The tube should be located as far as possible from elbows and bends and anything else that can disturb the smoothness of the flow; it should be in a long, straight part of the pipe, and since it is known that the velocity varies from point to point across the pipe a *traverse* of it must be made. One common way of making this traverse is to divide the circular cross section of the pipe up into five annular rings of equal area as shown in Fig. 49, the diameters of these rings being, respectively, 44.7, 63.2, 77.5 and 89.5 per cent of the pipe diameter. The tube is then moved across a pipe diameter, a reading of the velocity head being taken at each point marked with a dot and located as shown. From each velocity head the corresponding velocity is either calculated or read from such a diagram as Fig. 12, and where a fair degree of accuracy is desired it is advisable to traverse two diameters at right angles to one another and after the traverses are completed the average of all the velocities is the mean velocity in the cross section. Multiplying this mean velocity by the area of the pipe, in square feet, gives the discharge in cubic feet per second. (Note that it is the velocities and not the velocity heads that are averaged.)

As this process takes a lot of time it is rather more convenient to make the traverses once for all and by their aid locate the point in the section where the velocity is the same as the mean. The Pitot tube may then be set at this point and from its reading the mean velocity at that cross section may be read off the curve of velocity heads. It is possible that the setting of the tube may change with the mean velocity but usually no effort is made to check this point unless there is reason to suspect some variation.

When making the traverse it is always found that the velocity head varies across the pipe, but if the pipe is straight, with no cross currents caused by bends or obstructions, the maximum reading will be found very close to the center of the pipe. Often, however, the maximum reading is displaced from the center, and if this displacement is relatively large, it is usually advisable to move the tube to another cross section if possible, but if this cannot be done, the traverses must be carefully made at several discharges, where practicable, so as to locate the setting of the tube to obtain the mean velocity reading. Where the tube passes through a stuffing box it must be properly

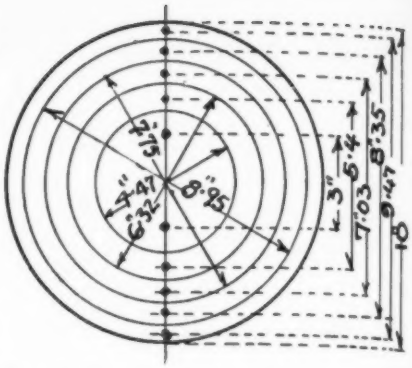


Fig. 49. Diagram of Method for Making a Traverse of Pipe to Obtain Mean Velocity

secured or it may be blown out of the pipe, and it is scarcely necessary to say that if the tube vibrates to any extent, its readings will not be reliable.

Since velocities, and therefore velocity heads, are usually small, it is quite general practice to use the inverted differential gage as shown in Fig. 33. If the upper part of the gage contains compressed air, the difference in levels *in feet* between the two water columns will actually be the velocity head at that setting and this is a very common way of working, but for very low velocities some such fluid as kerosene is often used above the water so as to get a magnified reading. Sometimes again, the gage with air on top is placed in an inclined position as shown in Fig. 50, where the slope is such as to give a magnification of five, but as the surfaces of the water in the legs of the gage are not horizontal, the gage is not always easy to read.

One trouble in the use of the tube, particularly for low velocity measurements, is caused by the presence of air bubbles in the connecting tubes between the gage and the Pitot tube, and as this has caused a number of false readings, it has had a tendency to discount the value of the instrument. A very little air may easily cause the gage to show a negative velocity head, which is, of course, sure proof of an incorrect application. The connecting tubes must be blow out from time to time and some of the readings should be checked.

A form of Pitot tube used a great deal in analyzing the flow in water works systems is shown in Fig. 51. In this tube both openings are affected by the velocity, the one pointing upstream being subjected to the sum of the pressure and velocity heads, while the one pointing downstream

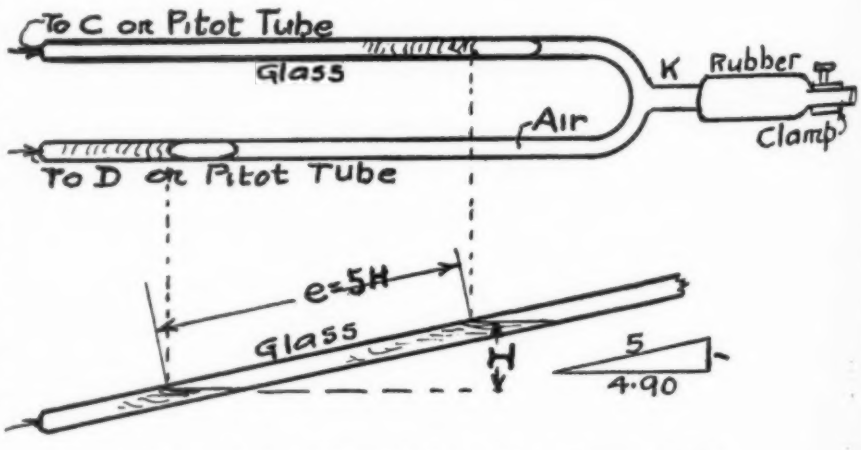


Fig. 50. Sloping Differential Gage with Air Above the Water

receives the pressure head less *part* of the velocity head. Tests show that a differential gage attached to this tube indicates less than twice the velocity head, and such a tube must always be calibrated in a stream of known velocity.

To make sure the matter is understood, suppose that an inverted vertical differential gage is used on a tube of the form shown in Figs. 47 and 48, and that the upper part of the gage contains kerosene, which weighs 0.82 times as much, per gallon, as water. If the gage reads 4.13 in., the velocity head will be

$$4.13 \times 0.18 \div 12 = 0.062 \text{ ft.}$$

as explained in Section X. That is $v^2/64.3 = 0.062$ or $v = 2.0$ ft. per sec.

In closed pipes the author has used both forms of the tube shown in Figs. 47 and 48 and has also used the first form successfully in open channels. In the latter case the gage is easily read by placing it at a convenient height and then sucking on the connection K of the differential gage till both columns appear in it; placing the gage on a slope gives good readings for low velocities. Either form of tube is readily made by a good mechanic; the first form requiring no comment, but the second one being more difficult to make. The lower part must be made from a solid bar, usually brass, and if it is used in a small pipe, the instrument is short and the holes are easily drilled up through it. For the longer instruments the author has joined the short lower end to the manometer connections by brass pipe through which he has run small brass tubes connecting the openings.

SECTION XV

Water Hammer

Up to the present this series of articles has dealt with hydraulic problems ordinarily met with in the water works field. Throughout, it has been assumed that the flow was steady and uniform, and that there was constant discharge, a condition always assumed in design. In operation, however, the flow is often variable due to the opening of a hydrant or even of a number of householder's faucets, although in the ordinary course of events the householder draws water at such a low rate that the system is little affected by the way the individual draws water. The house piping, on the other hand, is affected by careless operation of faucets which causes troubles and leaks in domestic plumbing.

But hydrants may draw very heavily on the distribution piping, as may also the supplies used by railroads, and if these are closed off rapidly dangerous pressure rises may occur. Under certain conditions the pressure rise is accompanied by a click, such as would result from striking the pipe with a hammer, and this phenomenon has received the name of "water hammer." An attempt will now be made to examine this matter briefly, although this must be done in a somewhat superficial way on account of the complexity of the subject.

In the first place it is to be noted that the pressure *rise* will depend upon the *rate* at which the valve or hydrant is closed and the velocity "extinguished." If, for instance, the velocity in a pipe supplying an open hydrant is 4 ft. per sec. and if this is reduced to 1 ft. per sec. by closing the hydrant then the pressure rise will

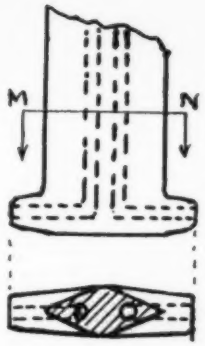


Fig. 51. Form of Pitot Tube Considerably Used in Water Works Systems

be greater if the closure takes one second than if two seconds are employed, in fact if the time taken to close the hydrant is long enough, there will be no appreciable pressure rise at all. Broadly speaking there is more danger to piping normally operating at low pressures than to those ordinarily under high pressure. Oddly enough, too, there may be quite as high pressure rise in short pipes as in long ones undergoing the same velocity changes. The only reason that there are rather more failures in the long pipes is that it is easier in these cases to have the conditions that produce the maximum pressure rise. As an instance of the severity of this effect in short pipes a case observed by the author showed a pressure rise of 90 ft. in a pipe 337 ft. long, carrying water at 3.9 ft. per sec. and in which the static pressure was only 54 ft. so that water hammer put the pressure up to nearly three times normal. This condition was caused by opening the switch on the motor without closing the pump discharge valve so that return flow was stopped by the check valve.

Water Is Compressible

So much has been said about water being incompressible that many people accept the statement as a literal fact whereas water is really an elastic fluid and will change its volume according to the pressure put on it. For example, if a 24 in. pipe 4000 ft. long is full of water at atmospheric pressure and it is desired to put on it a test pressure of 100 lb. per sq. in. then, if the pipe and fittings are perfectly tight and the material in the pipe itself does not stretch, it will be necessary to pump in approximately four cubic feet of water before this pressure will be reached. Obviously this shows that the water is compressible but only to such a slight extent that the change in volume may be neglected in the great majority of hydraulic problems. It is only because of this elasticity that water pipes do not burst every time a relatively large valve in them is closed. This elastic property is responsible for the interesting fact that when a valve is quickly closed the pressure rise is noticed at once on a gage on the pipe near the valve but it takes an appreciable time (about one second) before any change is noticed on a gage 4000 ft. up the pipe, the "pressure wave," as it is called, travelling up the pipe at a rate varying from 2500 to 4200 ft. per sec., depending on the size and thickness of the pipe and the material of which it is made.

The Pressure Wave

Limitations of space and the intricacy of the problems prevent a full discussion of the matter here although those wishing it might consult the author's book "Hydraulics for Engineers" or his paper "Water Hammer in Pipes, including those supplied by Centrifugal Pumps" which appeared in the Proceedings of the Institution of Mechanical Engineers (England) on page 245 of Vol. 136 (1937). The latter paper gives a full analysis of the subject and the periodical is in most large technical libraries. In ordinary water works distribution piping the velocity with which the pressure wave travels along the pipe, designated by a ft. per sec., for class C cast iron pipe is given in approximate figures in the following table:

Diameter of Pipe, in.	Velocity *a*-*ft.* per sec.
6	4200
12	4050
16	3960
20	3910
24	3880
30	3830
36	3800

As an example suppose there is a 12 in., Class C, cast iron pipe 8100 ft. long carrying water from a reservoir to some point below it where there is an open hydrant drawing 1200 U. S. gal. per min., corresponding to a velocity of 3.4 ft. per sec. in the pipe. Suppose, now, that the hydrant is quickly closed, then a pressure rise will be at once created in the pipe above the hydrant and this pressure rise will take 8100/4050 or 2 sec. to travel the full length of the pipe to the reservoir end, so that a gage on the pipe at the latter end would show no change in pressure for 2 sec. after the flow ceased at the hydrant. Both theory and experiment show that this pressure "wave" travels up to the reservoir, and when it reaches there it is "reflected" and travels down along the pipe till it comes again to the hydrant or valve, so that after the hydrant is

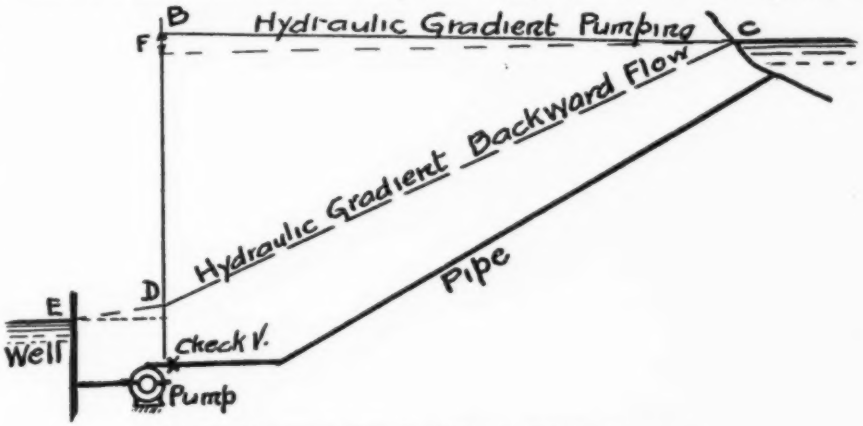


Fig. 52. Diagram of One Condition for Water Hammer.

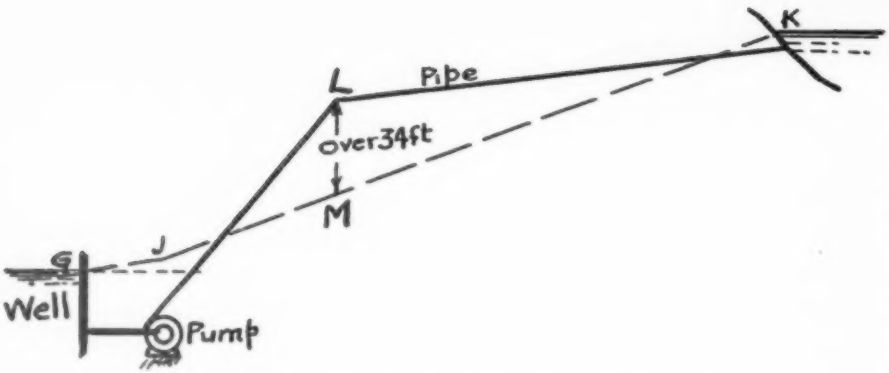


Fig. 53. Diagram of Condition for Backward Flow and Water Hammer.

closed a period of 4 sec. elapses before the returning wave reaches the valve or hydrant.

It is hardly necessary to labor the water works operator with these points for he knows that when a check valve closes against return flow there is an immediate rise in pressure at the valve, and for a brief interval the gage remains fairly steady, but shortly afterwards there is again rapid movement of the gage hand when the wave has again reached the valve.

The study of this action is fascinating but very difficult and it is only possible to mention certain features of it. The pressure rise depends on how quickly the flow is checked and the theory shows that if the closure of the outlet is completed before the return wave reaches it the maximum pressure will be produced; that is, in the case just quoted the maximum pressure will result if the hydrant is closed off in 4 sec. or less, but the pressure rise decreases rapidly as the time of closure is lengthened.

Inasmuch as the pressure wave travels at the rate of approximately 4000 ft. per sec. in cast iron pipe it would make the round trip in a pipe 400 ft. long in one-fifth of a second, a period much too short for the closure of an ordinary valve by hand although a check valve may close in less time than that; whereas in a pipe 40,000 ft. long the return trip would take 20 sec. which is sufficient time to close some valves mechanically. That is the reason why long pipes require special attention, because in them conditions producing maximum water hammer pressures are much more likely to occur than in the shorter pipes, but if the closure is within the period mentioned the pressure rise in the short pipe would be exactly the same as in the long one if the velocity had been the same in each.

Experiments

Our knowledge of this subject was vastly increased by the fine work of Joukouvsky, who conducted experiments in Moscow in 1897, on water hammer in pipes nearly 5000 ft. long, selecting pipes in the water works system for the purpose. He confirmed by experiment what he deduced by theory, that if the closing of a valve was completed before or at the same time as the return wave reached it then the pressure resulting had its maximum value, and the pressure *rise* in feet could be calculated from the formula

$$\frac{av}{32.2}$$

where v was the velocity in the pipe before closure began. Taking the value of a (the wave velocity) as already given for class C cast iron pipe it is evident that for this class of pipe the pressure rise may be about 120 times the velocity which was extinguished in the pipe if the closure was within the time given.

That is to say, if water is passing through a pipe at 3 ft. per sec. and is stopped, within the time limit given, by the closure of a valve or otherwise, there will be a pressure *rise* of 360 ft. at the valve. This rise will be the same whether the pipe is long or short and whether the normal working pressure is high or low, provided always that closure is completed when or before the return wave reaches the valve. (For longer closure times the pressure rise is lower.)

For a pipe designed for normal pressure of 30 ft. head quick valve closure will produce a total head of 30+360 or 390 ft., from the above data. This is 13 times the normal design pressure and would certainly be dangerous, whereas if the normal working design pressure was 360 ft. quick valve closure would only double the design pressure. For this reason special care must be taken in connection with low head pipes.

Pumping Lines

The pump discharge line of Fig. 52 illustrates one of the possible dangers from water hammer. When pumping, the hydraulic gradient is along the solid line *BC* sloping downward toward the reservoir. Suppose the power is suddenly cut off, the pump stops almost instantly unless it has a very large fly wheel effect, and the pressure at the pump drops to that due to the suction well, slightly below *D* on the drawing, so that the hydraulic gradient will take the dotted position *CDE* corresponding to backward flow in the piping. This will continue till the check valve closes, which often happens with great suddenness resulting in a pressure rise of magnitude given above.

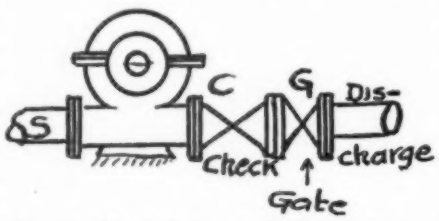


Fig. 54. Check Valves and Gate Valves on Pump Discharge.

A particularly bad case is that corresponding to the profile of the pipe shown on Fig. 53 where for backward flow the hydraulic gradient tries to fall more than 34 ft. below the profile as at *LM*. Since water cannot flow in tension the column of water in the pipe will break near *L*, as it is known to do in practice, and afterwards the two parts of the column, that starting at the pump and that ending at the reservoir, will reunite with great force, producing dangerously high pressure unless some means is taken of controlling it.

Air Chambers

This whole matter is fraught with so much danger and complexity that it is the work of an expert to figure out what possible pressure may occur in any case, particularly where the line is made of different sizes of pipe and has numerous branches. The author will, therefore, confine himself to the discussion of some methods that have been adopted to decrease the possible pressure rise. Undoubtedly check valves and some mechanically operated stop valves do sometimes close very rapidly and efforts have been made to slow up the water gradually when this does occur.

One of the best known methods in water works practice is the installing of an air chamber, which is often used in connection with pumping stations as well as in private dwellings. In pumping stations, the air chamber is placed near the discharge pipe check valve, and on the reservoir side of it, and the chamber must be at least as large in diameter as the pipe to which it should be connected by a sufficiently large opening. The chamber should be vertical and high enough to absorb the energy in the flowing water, but some form of air compressor must be connected to keep the chamber nearly full of air during normal pumping, and a gage glass should be installed so that the height of water in the chamber may be observed.

The function of the air chamber is, of course, obvious, for after the valve closes water flows up into it against a gradually increasing air pressure, so that the change of velocity is really very slow, and the consequent pressure rise correspondingly small. Its use in domestic plumbing is very common and generally very effective, where loose washers on faucets and lack of packing around the spindles often permit the valves to vibrate rapidly. When they are nearly closed this vibration may cause them to close and partly open alternately thus building up high pressures. Most people are only too familiar with this trouble, which shakes their pipes badly, and as a first precaution should see that there are no loose fittings. Quick closing taps are also often troublesome.

Special Valves

While the air chamber is in common use it is not an efficient way to solve the problem, because compressed air is not a good medium for storing up the energy in water as a result of its velocity. For the larger pipes special forms of valves are available and do their work effectively, being generally preferable to air chambers. To illustrate the matter, a centrifugal pump with suction pipe *S* and the beginning of the discharge pipe is shown in Fig. 54; it has the usual arrangement of valves, including a check valve *C* and a gate valve *G*, the latter usually being wide open during pumping.

In ordinary operation, the engineer usually closes *G* before cutting off the power so that the velocity in the discharge main is low when the pump stops, but if the circuit breaker opens without warning, or the power is suddenly cut off, then unless the pump is a very large one or has a big flywheel effect, it will stop pumping in a second or two, long before the operator could close the valve *G* even if he were close by when the trouble started. The water in the pipe will quickly lose its momentum and start flowing backward

through the pump, usually causing it to run backward, and the water quickly attains a high velocity so that when the check closes the velocity to be extinguished may be high. Reversal of rotation of the pump usually does no harm if the pump is made to stand it.

Should the check valve close just as the flow reverses at the pump the plan would be good, but there would still be a pressure rise nearly equal to the working pressure because the check has only been able to stop the end of the water column near it and the rest of the column up to the reservoir still has to be stopped since the water is elastic. Where the pipe is long a good plan is to have the check so controlled that it closes most of the way rapidly, finishing its closure very slowly.

In Section IX, an example was given to show that with the particular long pipe illustrated, a gate valve could be closed 77 per cent of the way (i.e. be left 23 per cent open) and still leave the pipe discharge 80 per cent of that at full gate. In other words 77 per cent of the valve movement could be made very rapidly without causing any great change in water velocity, and therefore without producing high pressure rise. While these figures illustrate the point they must not be taken as absolute and the exact effect must be worked out in each case separately.

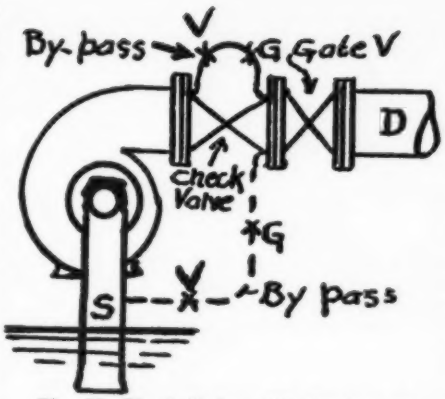
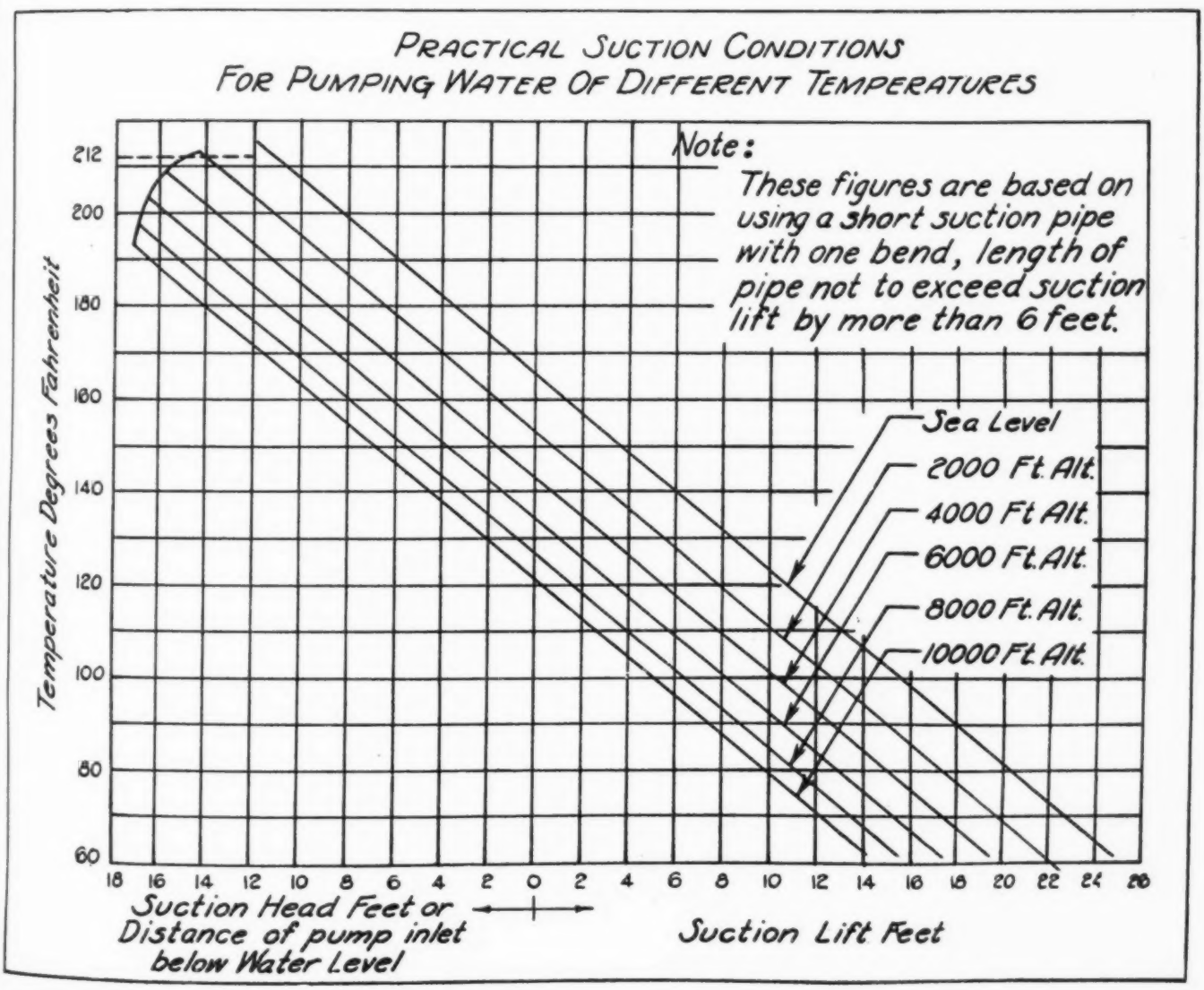


Fig. 55. Check Valve with By-pass on Pump Discharge.

In a long pipe line a regulated check valve could be used in which the first part of the closure could be quick and the last part gradual, but if the valve is a large one the slow part of the movement is hard to control and it may be advisable to use a smaller by-pass around the check, Fig. 55, with a controlled valve *V* in this by-pass. For instance a 36 in. check might have a 6 in. by-pass with controlled valve *V* and a gate valve *G* left wide open normally, and when reversal of flow began the large check valve might be of the quick closing type shutting off a large proportion of the water promptly. The by-pass could then be regulated to close at a slow enough rate to avoid serious pressure rise. If the by-pass is from the discharge side of the check to the suction of the pump, as shown by the dotted line, there will be little tendency for backward rotation of the pump, but, of course, the valve *G* would have to remain closed during pumping and automatically open instantly when the power was cut off. It should not be difficult to design such a valve.

Pumping Warm Water

For a particular temperature, the actual theoretical height that water can be drawn by suction by a perfect vacuum at sea level is obtained by subtracting the vapor pressure (in feet) of water at that temperature from 38.9 feet. For example, at room temperature (72° F.) this theoretical limit becomes 33 feet, and at 160° F., this limit is 22.9 feet. Since proper allowance must be made for water velocity and friction, the practical limits are less than the theoretical. If the water is hot enough, it is necessary to place the pump below the level of the water so that there is an actual head on the suction. The following chart gives practical suction lifts and suction heads for pumping water of different temperatures and at different elevations.


PRACTICAL SUCTION CONDITIONS
FOR PUMPING WATER OF DIFFERENT TEMPERATURES
Note:
These figures are based on using a short suction pipe with one bend, length of pipe not to exceed suction lift by more than 6 feet.
Sea Level
2000 Ft. Alt.
4000 Ft Alt.
6000 Ft. Alt.
8000 Ft. Alt.
10000 Ft. Alt.
Temperature Degrees Fahrenheit
212
200
180
160
140
120
100
80
60
18 16 14 12 10 8 6 4 2 0 2 4 6 8 10 12 14 16 18 20 22 24 26
Suction Head Feet or Distance of pump inlet below Water Level
Suction Lift Feet

MEASUREMENT OF WATER BY CIRCULAR ORIFICE AT END OF DISCHARGE PIPE

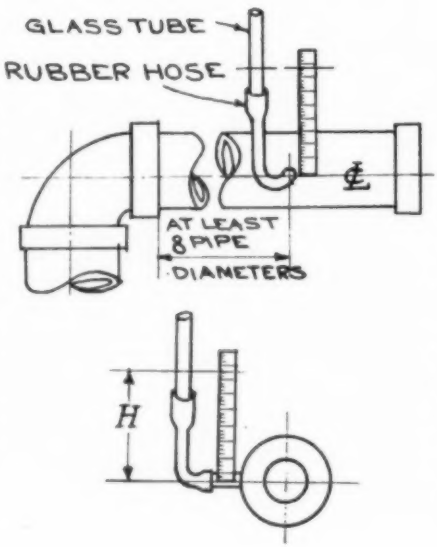
THE measurement of water flowing thru pipe lines by means of a circular orifice has been proven quite accurate, and experiments have demonstrated that a thin plate circular orifice installed at the end of a pipe line gives a rapid and accurate method for measuring such flows; the instruments being light and easily attached, permits of a simple method of testing pumping plants more accurately than the average weir that can be constructed in the field.

It is necessary that the pipe line be horizontal with no elbows, obstructions, or bends within 8 pipe diameters of discharge; the head should be measured from the center of the pipe at a point one foot or more back from the orifice.

The accompanying curve gives the discharge for 13 inch orifice on 15¼" inside diameter pipe, the formula being:

$$G = KA\sqrt{2gh}$$

Where G = Gallons per minute
A = Area of orifice in square inches



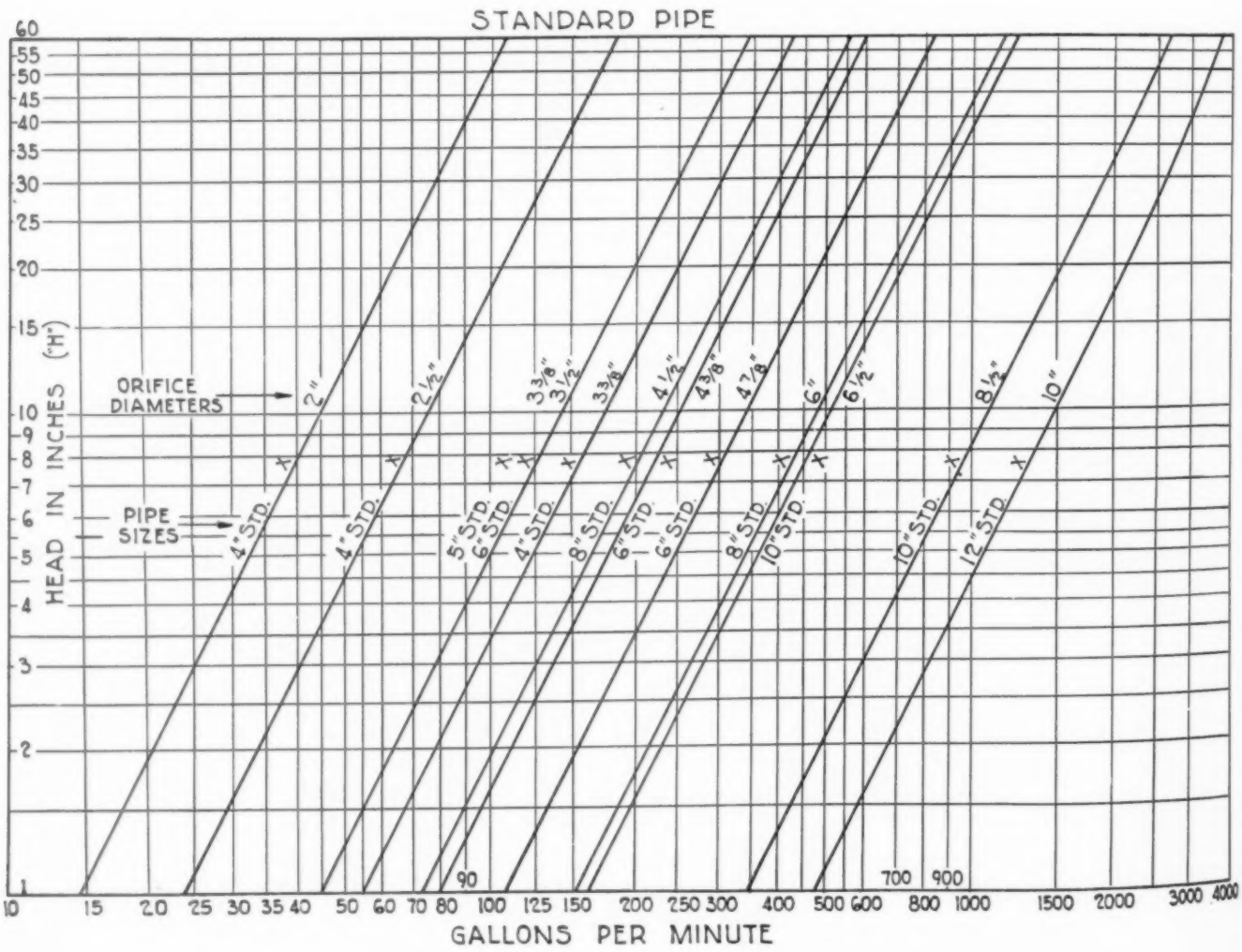
K = Constant depending on size of pipe and size of orifice.
H = Head in inches above center of discharge.

In calibrating these orifices a well constructed weir was used and the maximum variation of "K" was less than 3%. An average "K" was then taken from which these curves were constructed.

It is necessary that the orifice be running full of water and that "H" be at least one inch above top of the pipe for accurate results.

The head is measured by drilling and tapping for ⅛ in. pipe connection on the side of the discharge pipe one foot or more back of orifice, the hose connection installed and the height measured with an ordinary rule; the hole can then be plugged with an ⅛ in. pipe plug. It is very important that the ⅛ in. nipple does not extend inside the inner wall of the pipe and should be absolutely flush.

Peerless Pump Bull.



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Altitude Valves

Single Acting

Model 40 AWR — semi-throttling hydraulic control to prevent overflow of tank, standpipe or reservoir or to maintain a positive adjustable head for washing and filtering operations. Pilot adjustment range—4 to 30 ft., 30 to 60 ft., 60 to 150 ft.

Model 30 AWR—wide open or closed hydraulic control to prevent overflow of tank, standpipe or reservoir. Pilot adjustment range 4 to 30 ft., 30 to 60 ft., 60 to 150 ft.

Other single acting altitude valves combine back pressure or relief valve and pressure reducing functions.

Double Acting

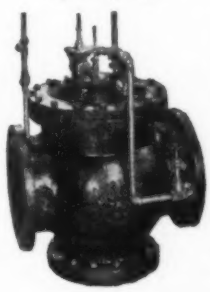
Model 40 DAWR—to prevent overflow in tank or reservoir and to allow return flow to distribution system when head on tank side is three inches higher than head of distribution system. These valves may be equipped with Remote Hydraulic and Electric Control. When required the return flow can be delayed for a predetermined pressure drop in distribution. Pilot adjustment range—4 to 30 ft., 30 to 60 ft., 60 to 150 ft.

Other double acting altitude valves combine many additional features to prevent dumping of tank, to retain supply on off peak loads and to make available maximum storage for peak load periods. Also a semi-throttling model for gradual closing of feed line and for maintenance of adjusted pressure on the distribution side. Can be equipped with secondary pilot to close valve if break occurs in distribution system.

Surge — Relief — Back-pressure Valves

Model 50 RWR—Single Pilot, Electric and Hydraulic Control—for protection against excessive pressures in distribution systems, supply lines and pump discharge lines. Intaliled in discharge line of deep well pump, provides constant back pressure to prevent overdrafting of the well. Pressure range in pilot—10 to 60 lbs., 20 to 150 lbs., 125 to 250 lbs.

Float Valve or Level Controller



Model 50 FWR—External Pilot Operated—to control main valve so that main stem operates wide open or closed. Pilot valve wastes to atmosphere contents only of operating chamber each time valve stem rises to open position. Maximum inlet pressure, 175 lbs.

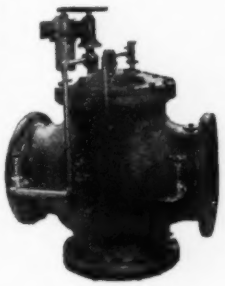
Model 40 FWR—External Pilot Operated, Semi-throttling—for gradual closing of valve during last three inches before overflow.

Reducing and Regulating Valves

Model 40 WR—Single Pilot, Hydraulic Control—to maintain constant desired discharge pressure regardless of rate of flow or pressure on upstream side of valve. As a pressure reducing valve, operates to:

1. Control and regulate pressures in gravity and pumping systems.
2. Regulate flow between reservoirs and zones of different pressures.
3. Regulate fire flows between zones of unequal pressures.
4. Regulate pressures in filter wash lines and aerator nozzles.

Valves operate without waste of water to atmosphere and will not induce surge. Can be equipped with dual pilot to permit changing from one pressure to another by mere operation of a three-way cock for systems operating under different day and night pressures. Valves may be supplied with right-angle body. Pressure range outlet—10 to 60 lbs., 20 to 150 lbs., 125 to 250 lbs.



Model 40 WR—Single Pilot, Electric Remote Control—motor driven, geared to pilot adjusting tail screw, made with limit stops to prevent over-travel, and operated by push button switch.

Pressure Reducing and Reverse Flow Valves

Model 50 WR—Single Pilot, Hydraulic Control—for installation at dividing line between two zones to reduce pressure from higher zone, to function as a sleeping valve, discharging into low zone at a time of excessive draft, to permit flow from low to high zone when excessive draft in high zone reduces its pressure below that of the low zone as in case of fire. Pressure range outlet—15 to 60 lbs., 20 to 150 lbs. This valve may be equipped with dual pilot control to permit greater ranges in pilot control and to provide the assurance which goes with duplication and stand-by units.

FIRE DEPARTMENT SPECIALTIES

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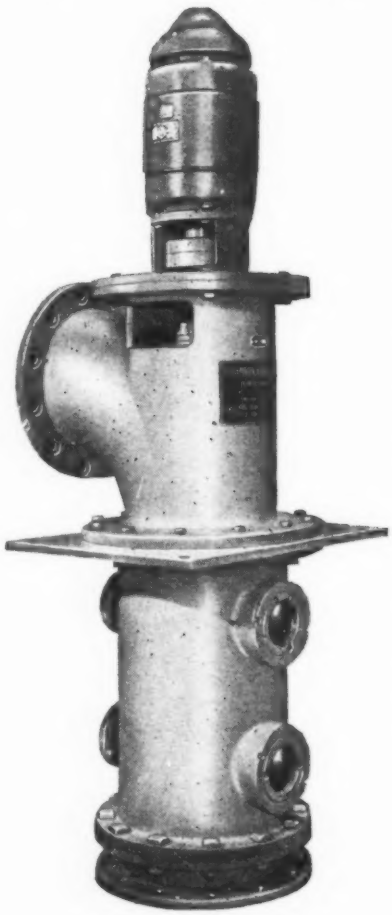

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POSITIVE FLIGHT CONVEYORS—for rectangular settling tanks.

CIRCULAR CLARIFIERS—for round settling tanks.

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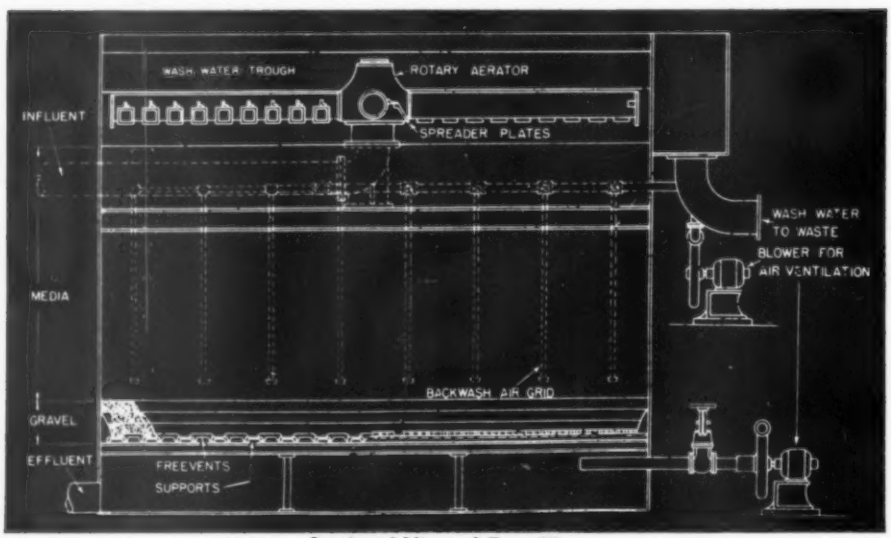

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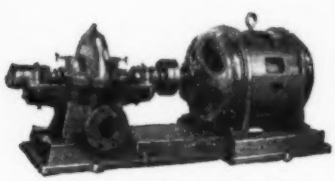


Horizontal Double Suction

CAVITATION IN PUMPS

By ROBERT W. ANGUS
Consulting Engineer, Torono, Ont.
(Professor Emeritus of Mechanical Engineering, University of Toronto)

THE destructive action of swiftly flowing water on its containing channels and on engineering structures has been noticed by even the most casual observer who has looked upon the mighty gorges carved by rivers in solid rock, the damaging action of water on bridge piers and on dams, or the destruction brought by waves. In natural channels, and on concrete structures such as dams, erosion is frequently due to abrasive materials and to large rocks and tree trunks transported by the water. Large rocks often batter the outer surface off concrete and allow the water to dig out the materials below, and heavy tree trunks break the structures and allow destructive eddies to form as turbulent water is quick in its deteriorating effects.

The Pitting of Metals

In this article, however, it is not this important problem that is being discussed, but another matter which more particularly affects the municipal engineer and his water or sewage plant, for machinery also is subject to the same type of damage as the concrete structures. Propellers of boats are often very badly pitted, and lose much of the metal in them, and at least one case is known where the propeller of a destroyer was so badly damaged by water that it had to be replaced after only a few hours running at full speed, and this in a case where the ocean water was the only source of trouble.

All engineers having to do with water turbines are aware of the fact that holes often appear in the metal, particularly in parts of the runner or rotating element, these holes frequently becoming so deep that the machines have to be shut down and the runners built up, or replaced at great expense. This shutdown also entails a great loss of power output on account of the long time needed for repairs.

The municipal engineer is much more familiar with the difficulties occurring in valves and in centrifugal pumps, where the water is usually clear and free from harmful chemicals, and the troubles cannot be ascribed to ordinary abrasion or to chemical action. The slower speed pumps ordinarily give little trouble, but the use of high speed motors and direct connected pumps has brought the matter forcibly forward.

As an illustration of what may happen in pumps, the photograph shown in the top of Fig. 1 has been made from part of the shroud cut out of the suction side of an old cast iron impeller, this part being removed from the rest so as to make a good photograph. The shrouds of this impeller were approximately ⅜ in. thick and the photograph shows that holes were eaten entirely through the metal, and that large areas were affected, mostly at the discharge ends of the vanes. The vanes themselves have been partially destroyed and, while this is a very old type of impeller, it shows what can happen in a short period. Incidentally, this was one of the impellers, tried many years ago and later discarded, where half of the vanes only extended half way in, and it is evident that the water decided these short vanes were harmful and removed them, leaving only a faint trace of them to be seen.

The bottom portion of Fig. 1 shows a turbine runner damaged by cavitation.

Causes of Damage

At the outset it should be stated that damage to pumps may be due to several causes. In mines the pumps often handle very abrasive materials, and the same is true of pumps used on suction dredges, while in chemical plants the liquids pumped may even dissolve the metals. These phases of the subject seem to be more easily solved than is the case where the liquid is clear and free from harmful chemicals. Assuming the pumps otherwise correctly designed, the protection against abrasives and chemicals is primarily affected by the use of suitable materials in the pump parts. The Standards of the Hydraulic Institute, and also a number of books and catalogues, furnish lists of metals suitable to the different cases. Sometimes even rubber linings are used to provide against wear, and builders have been fairly successful in overcoming wear from abrasion, although, in some cases, even with the most suitable metals, wear is so rapid that convenient and relatively cheap replacement of the worn parts is the only recourse.

Generally speaking, however, water turbines and municipal pumps have to do with clear cold water, as was true for the pump shown in Fig. 1, and yet the damage goes on and causes much anxiety and trouble to those responsible for operating them. The deterioration of turbines has caused most concern, because of their costly nature and the great loss of power involved when they must be shut down for repairs, for they must often be out of service for weeks while being reconditioned. The damaged part may have to be completely replaced or the holes welded up with stainless steel or other material at great expense. While water turbines have always been subjected to the damaging effect of the water, it was really the extended use of the low head, high speed reaction machines that made it most acute. Of recent years many protracted studies have been made on these turbines, by getting data from existing plants and by setting up expensive laboratories in which various types of experiment could be made.

Cavitation—What Is It?

These facts are mentioned because it is very largely from these turbine experiments that we have learned what to do with pumps. The turbine and pump are identical in principle, subject to the same laws of flow and to the same "diseases." The turbine studies showed that high speed machines were more affected than the slower rotating ones, and also that the trouble was greater the higher the machine was placed above the tail water. The action is undoubtedly due to mechanical hammering of drops of water against the metal, thereby eroding the metal or causing cavities, and this action has been given the name of "cavitation."

Strangely enough, cavitation damage does not occur in parts of the machines where the water has a positive pressure against its containing channels, but so far

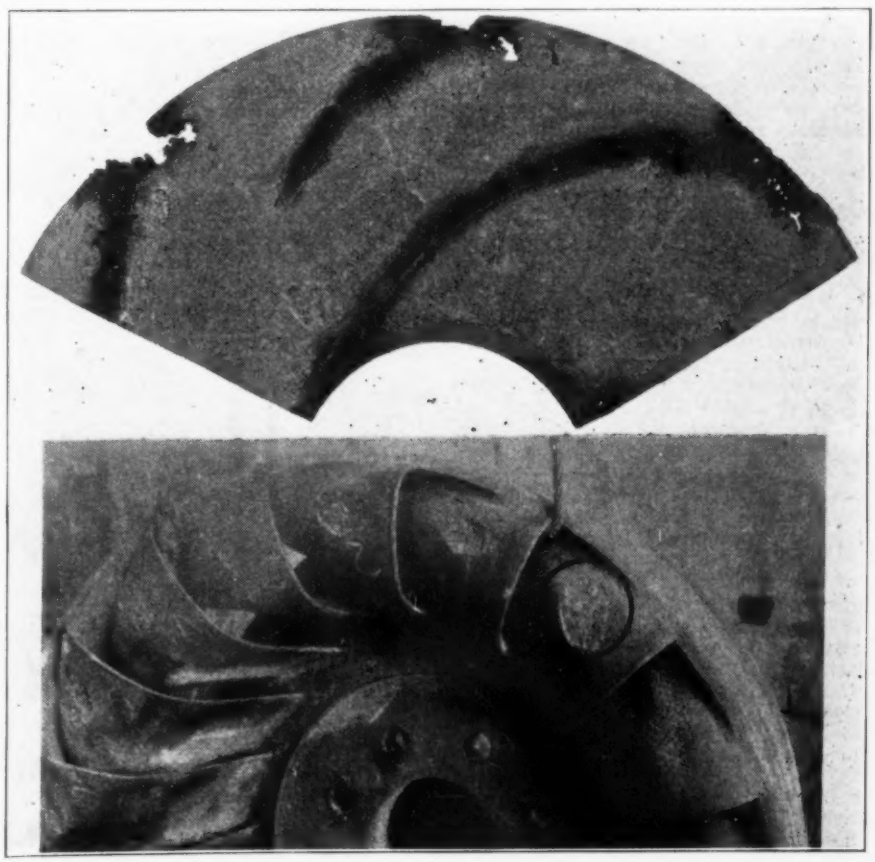


Fig. 1. Damage Done by Cavitation; Top—Part of an Impeller; Bottom—Turbine Runner. The part inside the circle has been chipped out before welding repairs.

as can be observed, its effect is produced on surfaces where the water pressure is low, often much below atmospheric, and where there may be actual minute separation between the actively flowing water and

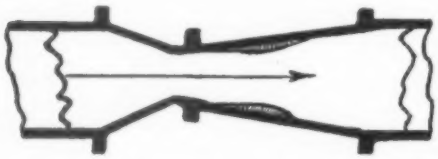


Fig. 2. Venturi Channel Used in Cavitation Studies

the metal. If there is large separation, eddies may form and another kind of action results.

The mechanism of cavitation is not fully understood, although photographs with extremely high speed cameras have been taken through glass-sided models, and many turbine and pump builders have a cavitation stand where glass windows may be inserted in the model and the action observed by means of a stroboscope.

To give a definite picture of the process, a Venturi tube, Fig. 2, may be used as an illustration inasmuch as the Venturi tube has been very largely used in cavitation studies. The tube illustrated has been shown, for clarity, with an exaggerated taper on the down stream end and, as used in most studies, it has a rectangular throat, instead of a round one; the up and down stream ends are also rectangular with all the rectangles of uniform width, so that the taper is all in one plane; this allows the tube to be made with glass sides.

Picture of Cavitation

For low velocities in the tube, and up to those in ordinary use with the Venturi meter, observations through the glass sides show that the water completely fills both ends and rubs against the walls. But as the velocity increases, a point is reached where the water is unable to expand and completely fill the down stream end, as illustrated, and a foggy area, hatched in the figure, is observed to form above and below the stream, and a little downstream from the throat. Simultaneously with the formation of the foggy area, a noise is heard inside the tube, this noise giving the impression of pieces of shot being projected against the tube wall. If the walls have initially been polished, a few hours' running shows that, in the affected area, the wall surface has become dull, and in prolonged runs will be definitely pitted, even though the metal is hard.

In the tube there is, of course, a decided lowering of pressure near the throat, and high vacua may easily be produced there, although they never reach that of the barometer. The following explanation given of the phenomena seems to be reasonable. The flowing water contains, naturally, a small volume of air and often other gases, and these are probably in the form of small bubbles. In the upstream end these bubbles, which doubtless contain water droplets, are under pressure and of small volume, but as they approach the throat at high velocity, the pressure falls rapidly and the bubbles expand, and if the pressure is low enough, they burst with explosive violence, the contained particles of water flying out against the walls and producing almost incredibly high pressures, as the measurements show. These blows are so hard that no metal can stand against them, although ductile metals like steel resist much better than cast iron. The noise is rather distressing and is the first indication the operator has that cavitation is present in the ordinary machine. While in this case there is clear separation of the water and walls, similar experiments show that the same action occurs without visible separation in the regions of low pressure.

Photographs and direct observation bear out the probable correctness of this theory, which is further confirmed by the fact that damage in pumps is much more common in regions of low pressure than elsewhere. High suction lift is to be avoided, as it produces low pressure in the eye of the pump. But this alone is not sufficient, for the pressure at the inlet end of the impeller vanes is affected by the general shape of the inlet and the rapidity with which the water changes its direction and velocity at entry. Anything causing a rapid acceleration of the water, or a quick change in its direction, will evidently result in a lowering of pressure and a tendency to cavitation. High speed pumps are in more danger than those of lower speed and badly shaped vanes, particularly at the entry end, cause a very quick change in the direction of flow and resulting acceler-

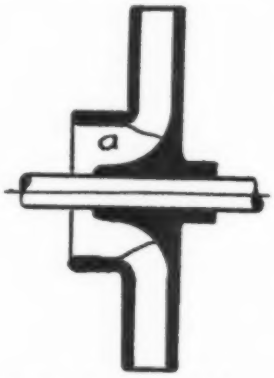


Fig. 3. Single Entry Pump of Low Specific Speed.

ation of the water with consequent pressure drop. Volatile liquids are more difficult to handle, from this point of view, than heavier ones, and very careful consideration must be given all these points in every case.

Relation of Pump Design to Cavitation

A section of a single stage, single entry centrifugal pump is shown in Fig. 3. Considering the eye of the pump and entry to the impeller, it is seen that, where the pump is set above the well, there will be a low pressure, *a*, due to this elevation, and this pressure is further lowered by the velocity head at *A* and the resistance loss in the suction pipe up to this point as well as by the vapour pressure of the liquid.

Undoubtedly, the general proportions of the impeller have an effect on this phenomenon. In large capacity, low head pumps, the entering velocity and the angle through which the water turns as it enters the impeller are usually greater than in high head small capacity machines, and more care must be taken in setting the former. Whatever the explanation, the proportions and general shape of the impeller do have a marked influence on the tendency toward cavitation.

Cross-sections, along the shaft axis, of a few typical forms of impellers are shown in Fig. 4, that at the left being for small discharge and high head pumps, while progressing toward the right hand side there is finally shown a form for low head and large discharge. While it is not desirable here to go into the mathematical proof, it may readily be shown that the shape and general proportions of the impeller are indicated, curiously enough, by the numerical value of the expression

$$\frac{N\sqrt{G}}{H^{0.75}}$$

or what is briefly known as the specific speed, where N is the rotative speed rpm., G the discharge, and H the head in ft.

Unfortunately, there is confusion of units with regard to the discharge, which is expressed here in United States gallons per minute, but sometimes Imperial gallons are used, and less frequently cfs.

In Fig. 4, the left hand pump has the lowest and the right hand pump the highest specific speed of the three.

The Cavitation Constant, Sigma

Experience has shown that the height of the suction lift and the shape, as indicated by the specific speed, are the two main factors entering into the cavitation problem. Largely as a result of his studies on water turbines, Thoma has suggested that the limiting suction lift, for each specific speed, depends on the velocity of the water entering the impeller, on the elevation of the pump, and on the barometric pressure corrected for the vapor pressure of the fluid being pumped. Thoma's formula,

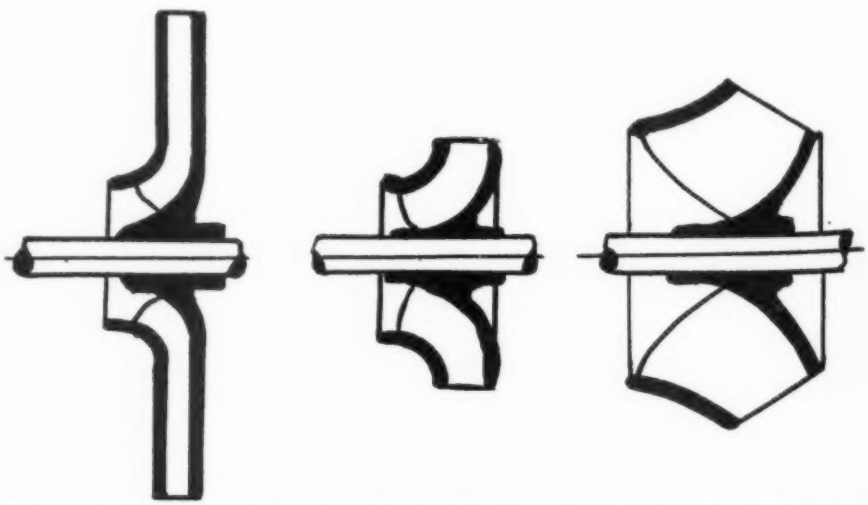


Fig. 4. Impellers of Different Specific Speeds. Low N_s on Left; High N_s on Right.

which has been adopted very generally is

$$\text{Sigma} = \frac{B-h}{H}$$

where B is the barometric pressure reduced by the vapor pressure of the liquid being pumped, h is the height of the center of the impeller above the well surface, plus friction loss and velocity head at the entry to the impeller, and H is the head pumped against; all the quantities are in feet and feet of water column. If the barometer reads 30 in. and clear water at 70 deg. F. is being pumped (for which the vapor pressure is 74 in. mercury) then the corresponding value of B is (30—.74) x 13.6/12=33.1 ft. of water.

This number, *sigma,* is called the cavitation constant, the numerator $B-h$ is the pressure or vacuum due to the effective lift and the value of sigma is the same whatever units are used, so long as the three quantities are in the same units. Evidently the maximum elevation of the pump corresponds to $h=B$ ft., for which sigma is zero, and the larger the values of the coefficient the lower the pump must be set, and it may be at or below the well level.

Experience of many pump builders in America has been combined, and as a result of data submitted by 24 of them, the Hydraulic Institute of New York has plotted a series of curves from which Fig. 5 has been prepared; this diagram shows the approximate relation between the cavitation coefficient and the specific speed for double suction single stage pumps, but since the constant depends, to a small extent, on the head as well as on the specific speed, the curve is an average for all heads. Similar data are available for other types of pumps. This curve in Fig. 5 shows the minimum value of sigma permissible for safe operation with minimum danger of destructive cavitation; all pumps set with larger sigma (smaller suction lift) are likely to have little trouble if they have been properly designed and built.

Relation of Sigma to Ns

As an illustration, a double suction pump for 10,000 gpm (U.S.) at 32 ft. head and 1400 rpm, has a specific speed of 10,000 and would have sigma about equal to unity. If $B=32$ ft., this pump could not have any suction lift and would have to be set below well level to provide for velocity head and resistance. Had the pump been designed for the same conditions but running at half the speed, its specific speed would be 5,000 and the corresponding sigma would be .37; the possible suction lift

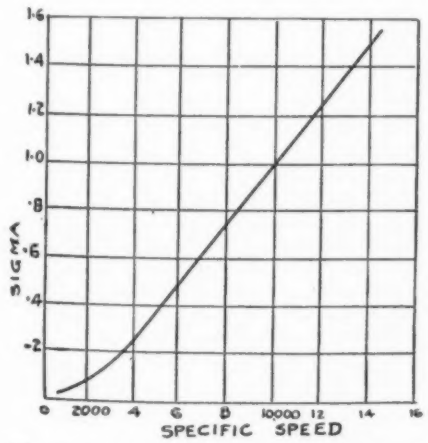


Fig. 5. Relation Between Specific Speed (N_s) and Possible Suction Lift, as Given by Sigma, for Double Suction Single Stage Pumps.

allowing for friction, etc., would be increased to over 20 ft., permitting the pump to be set at least 16 ft. above the well.

An examination of Fig. 5 shows that sigma increases rapidly with the specific speed, that is, pumps of very large discharge and low head, running at high speed (such as pumps for lifting raw water on to filter beds or for circulating water in condensers) have a large value of sigma and must, therefore, have a small positive and often a negative value of h. This means that if cavitation is to be avoided, the pumps must be set close to or actually below the surface of the well. The high head, low discharge pumps may, on the whole, be set much higher. Evidently the lower the speed the less is the danger from cavitation so that, from the cavitation standpoint, low lift pumps particularly should run at as low a speed as possible.

Pumps at or near the sea level may run with higher suction lift than those at higher elevations, and hot water and fluids which vaporize at low temperatures must have a correspondingly low suction lift. Cavitation is more destructive to cast iron impellers than to steel or bronze, although experiments show that the grade of bronze has a very marked effect.

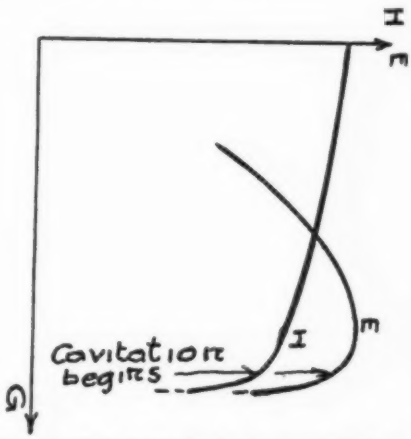


Fig. 6. Effect of Cavitation on Efficiency and Discharge.

In computing the specific speed, the discharge and head usually chosen are those corresponding to normal conditions and should, therefore, be those of best efficiency. As pumps are often run at other discharges and heads, it is evident that for these discharges there is a different sigma than for best output. For smaller discharges than normal the specific speed will be decreased and this will correspond to a lower sigma or higher suction head, so that in general the pump is safer from cavitation when running under reduced discharge, although if the discharge is reduced sufficiently there may be some danger of it due to the bad entry and exit conditions in the impeller.

On the other hand, increased discharge renders the pump more liable to cavitation trouble, and practically every pump suffers from it at the very large discharges, that is when running much below best head.

Effect of Cavitation on Pump Efficiency and Discharge

The operating engineer is first aware of cavitation by the noise in the pump and usually vibration accompanying it, but it may easily happen that serious damage may have been done before the noise is disturbing. Probably the best indicator, outside of direct examination of the parts, is a study of the characteristic curves for

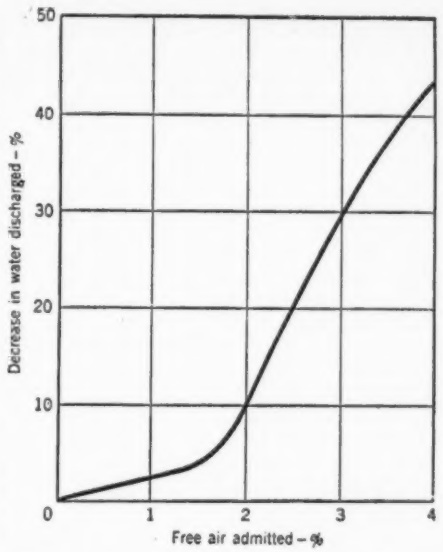


Fig. 7. Decrease in Discharge of Centrifugal Pump Due to Admission of Air Into Pumps

the pump, which normally have the general shapes shown in Fig. 6 within working ranges of discharge. The head curve H and the efficiency curve E slope gradually downward. When cavitation begins there is a definite lowering and increase in steepness in the efficiency and discharge curves, the former being the more marked. This is indicated in Fig. 6. Unfortunately, the operator has no exact method of making frequent plots of these curves.

Some experimental pumps have been made with glass sides and the action of the water examined with a stroboscope. The information gained was most valuable and included data on suction conditions, and it was observed that vapor appeared at the outer periphery of the eye of the pump before any change in discharge occurred, particularly for high specific speeds. Of course this air reduces the available inlet area and induces cavitation, so that if cavitation is suspected the pump should be carefully examined, particularly for the larger sized machines. When hot water or oil is being pumped the danger of pitting increases.

Where pumps are operated at high altitudes there must, naturally, be a lower suction lift than if the same pump is working at or below sea level.

Noisy Pump Indicates Cavitation

A noisy pump indicates cavitation, but not always of a very destructive nature. Not long ago the author examined a very large pump working on water supply, but at low head because it was delivering water from a well to the filter beds. This pump was exceedingly noisy when operating at rated discharge, and yet it was a very efficient machine. The noise was so bad that conversation was impossible anywhere near it and to obviate this difficulty, one or two small holes, little over ⅛ in. diameter, were drilled in the suction pipe close to the eye of the pump. This plan was most effective, almost completely eliminating the noise and making no appreciable change in discharge. Although now for some years in service, this pump shows no deterioration. A word of caution, however, for experiments show that very little air may affect the discharge greatly. As shown in Fig. 7 an amount of 5 per cent by volume would reduce the discharge by many times this per cent.

GALLONS IN WATER SUPPLY PIPE*

Water supply men often want to know the volume of a given length of pipe in gallons. This writer does not know of any table that gives such values and where volumes are to be found they are usually given in cubic feet. Besides, the range of a table is seldom great enough to cover all desired conditions.

This chart takes care of all standard pipe sizes from ⅛ inch to 15 inches on scale A, and any length of pipe line from 0.1 foot to 800,000 feet on scale B. Scale C shows volumes from 1 to 1,000 gallons.

In using the chart simply run a straight line through the pipe size (column A) and the length of pipe line (column B) and the intersection with column C immediately gives the total number of gallons in the pipe or pipe line.

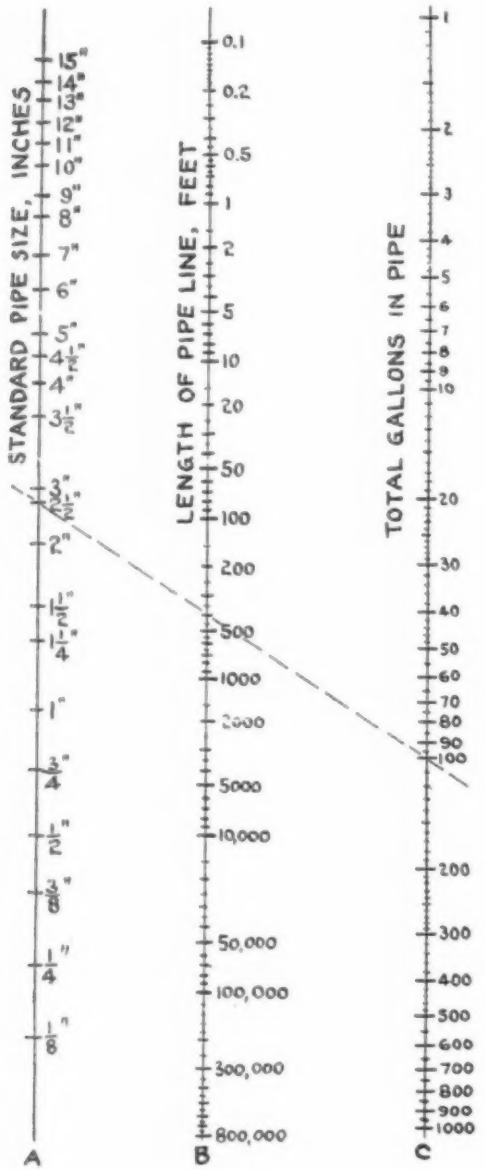
Thus the dotted line drawn across this chart shows that if the pipe size is 2½ inches (column A) and the length of line is 400 feet (column B) the volume of the pipe (column C), is 100 gallons.

For a length of pipe which is found to hold more than 1,000 gals. the volume is readily found for 1/10th or 1/100th of the length and the C scale reading multiplied accordingly.

Or, if it is desired to know the length of pipe line required to hold a given number of gallons, or the size of the pipe necessary to hold a given number of gallons within certain limitations of length, the chart may be conveniently applied. In other words, knowing two factors in any two of the three columns, the unknown in the third column is quickly found.

Here is another "trick": To determine the volume of a ½-inch pipe one foot long, the easiest way is to run a line through the ½-inch, column A, and the 10,000 in column B. The intersection with column C says 160 gallons, but that, of course, is for a pipe line 10,000 ft. long. By merely pointing off four places to the left we have 0.016 gal, as the volume of a ½-inch pipe one foot long.

* Contributed by W. F. Schaphorst (M.E.), Newark, N. J.



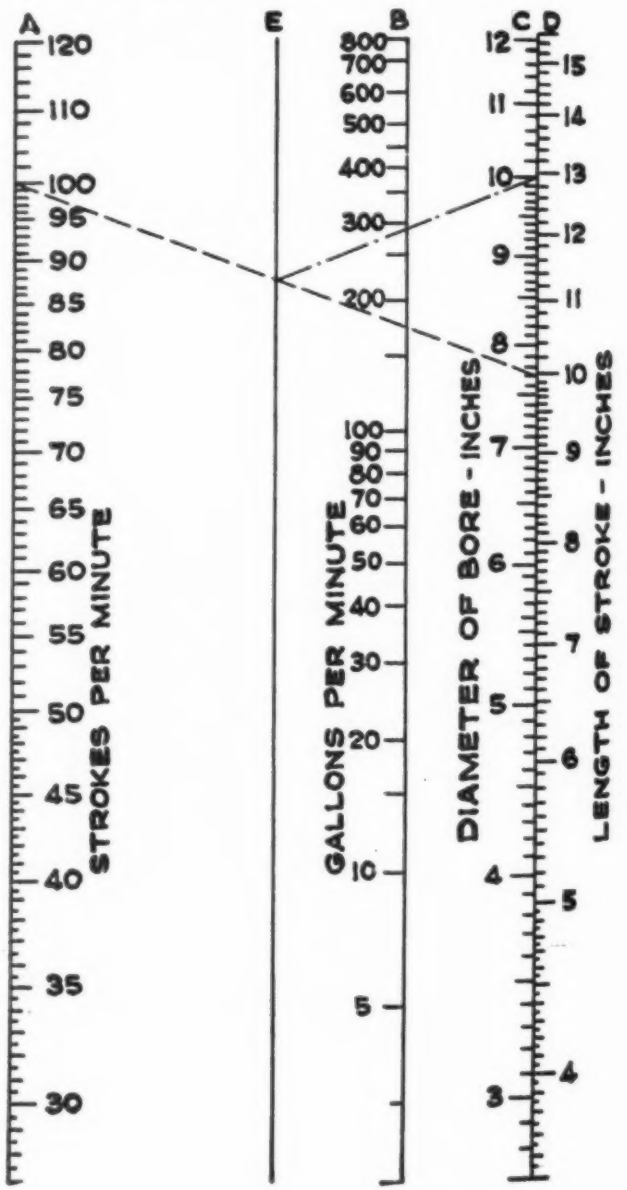
CAPACITY OF RECIPROCATING PUMPS

This nomograph shows the capacity of a reciprocating pump operating without slip where the size and number of strokes is known.

Example:

A pump of 10-in. bore, 10-in. stroke running 100 strokes per minute: Locating 100 on line "A" and 10-in. on line "D" we draw a line between them; then locating the intersection of this line with line "E" and locating 10-in. on line "C" we draw a connecting line. Where this line crosses "B" we have the delivery, or 290 G.P.M., of the reciprocating pump.

(From "Data Handbook" of Economy Pumps, Inc.)



SURVEY OF ELEMENTARY MATHEMATICS

For Water and Sewage Works Operators

By WARREN K. EGLOF, Ph.D.
Head, Dept. of Chemistry
Niagara University, N. Y.

The operation of water and sewage works requires no advanced mathematical operations, but the proper use of elementary mathematics will greatly improve the value of laboratory and operating data. This article on the fundamentals of elementary mathematics, we believe, will be of help to all water and sewage works men regardless of training, education, or experience.—The Editors.

Fractions

A FRACTION is an expression of unexecuted division. In itself, it is one or more of the equal parts of anything. The *terms* are the numerator and denominator. *Similar* fractions have the same denominator. An *integer* expresses whole units.

Multiplication

To multiply a fraction by an integer, multiply the numerator of the fraction by the integer and place over the same denominator.

$$5 \times \frac{3}{7} = \frac{15}{7}$$

To multiply a fraction by a fraction, multiply the numerators for the new numerator and multiply the denominators for the new denominator.

$$\frac{2}{3} \times \frac{5}{7} = \frac{10}{21}$$

Division

The reciprocal of a number is one (1) divided by that number. To divide an integer by a fraction, multiply the integer by the reciprocal of the fraction.

$$\frac{5}{\frac{2}{10}} = 5 \times \frac{10}{2}$$

To divide a fraction by a fraction, multiply the dividend by the reciprocal of the divisor.

$$\frac{\frac{2}{3}}{\frac{4}{7}} = \frac{2}{3} \times \frac{7}{4} = \frac{14}{12}$$

Addition and Subtraction

The *factors* of a number are all the numbers that multiplied together produce the given number.

$$2 \times 3 \times 5 \times 9 = 270$$

The numbers 2, 3, 5, and 9 are a set of factors of 270. A *multiple* of a number contains that number as a factor. A *common factor* of two or more numbers is a factor of each.

Only similar fractions can be added or subtracted. To add, add the numerators and place over the common denominator.

$$\frac{2}{5} + \frac{1}{5} = \frac{3}{5}$$

Dissimilar fractions must be reduced to similar fractions. This is done by finding the lowest common multiple of the denominators. The denominator of each fraction is divided into the lowest common multiple to get the factor used to reduce the given fraction. The following example illustrates the rule.

$$\frac{2}{3} + \frac{4}{7} + \frac{5}{9}$$

63 is the lowest common multiple—i.e., the lowest number into which all three denominators will go an even number of times.

$$\frac{42}{63} + \frac{36}{63} + \frac{35}{63} = \frac{113}{63}$$

Cancellation

If the numerator and denominator of a fraction are multiplied or divided by the same number, the value of the fraction is not changed. This is the principle of cancellation.

If, however, the same number is *added to* or *subtracted from* both the numerator and denominator of a fraction, the value of the fraction is changed: For example—

$$\frac{3}{4} = \frac{3 \times 2}{4 \times 2} = \frac{6}{8}$$

but $\frac{3}{4}$ is not the same as $\frac{3+2}{4+2}$ which is $\frac{5}{6}$ and not $\frac{6}{8}$ or $\frac{3}{4}$

Decimals

When anything is divided into tenths, hundredths, etc., the parts are called decimal divisions. One or more of the decimal divisions of a unit is called a decimal fraction. The denominator of a decimal fraction is not expressed but is indicated by the position of the digits in reference to the decimal point.

$$0.1 = \frac{1}{10} \qquad 0.543 = \frac{543}{1000} \qquad 0.00062 = \frac{62}{100{,}000}$$

Locating the Decimal Point

Annexing zeros to a decimal does not alter its value for calculation. In 0.00624 the two zeros between the point and the six serve to locate the decimal point and have no other meaning.

Common Fractions into Decimals

A common fraction is reduced to a decimal fraction by dividing the numerator by the denominator.

$$\frac{2}{5} = 0.4 \qquad \frac{9}{16} = 0.5625 \qquad \frac{2}{7} = 0.2857 \text{ - - - etc.}$$

Finite and Infinite Decimal Fractions

A finite decimal fraction expresses the value of a common fraction exactly. If the denominator of a common fraction contains no other factors besides 2 or 5, it will give a finite decimal fraction; otherwise, an infinite decimal fraction results, as, for example—

$$\frac{1}{7} = 0.142857142857 \text{ - - - etc.}$$

Rounding Off

In practice, infinite decimal fractions must be rounded off. The operator must decide how many decimal places are to be retained. The others are discarded according to the following rules: If the part discarded is greater than five, increase the last place retained by one unit. If the part dropped is less than five, discard without further change. If the part discarded is exactly five, increase the last place retained by one unit.

$$\frac{1}{7} = 0.14285714 \text{ - - - etc.}$$

$$= 0.14$$
$$= 0.143$$
$$= 0.1429$$
$$= 0.14286$$

Converting Decimals to Fractions

To change a decimal fraction to the nearest number of eighths, sixteenths, etc., divide by the decimal equivalent of ⅛ or 1/16. For example—

$$0.34375 = \text{how many } \frac{1}{32}\text{'s}$$

$$\frac{1}{32} = 0.03125 \text{ and } \frac{0.34375}{0.03125} = 11 \text{ or } \frac{11}{32}$$

Placing the Decimal Point

The number of decimal places in a product is the sum of the decimal places in the factors. The number of decimal places in a quotient is equal to those in the dividend minus those in the divisor.

Scientific Notation

$$\begin{aligned}
10^0 &= 1 \\
10^1 &= 10 \\
10^2 &= 100 = 10 \times 10 \\
10^6 &= 10 \times 10 \times 10 \times 10 \times 10 \times 10 \\
0.1 &= 10^{-1} \\
0.01 &= 10^{-2} \\
0.0001 &= 10^{-4} \\
0.00473 &= \frac{473}{100{,}000} \\
&= \frac{47.3}{10{,}000} \\
&= \frac{4.73}{1000} = 4.73 \times 10^{-3}
\end{aligned}$$

Any number may be expressed as a number between 1 and 10 multiplied by some power of 10.

$$92{,}000{,}000 = 9.2 \times 10^7$$
$$349.2 = 3.492 \times 10^2$$
$$0.00498 = 4.98 \times 10^{-3}$$

The power of 10 used is equal to the number of places that the decimal point has been shifted; positive if moved to the left and negative if moved to the right. Numbers in the scientific notation may be added or subtracted only when the powers of 10 are equal for both.

$$3.72 \times 10^3 + 8.56 \times 10^3 = (3.72 + 8.56)\ 10^3$$
$$= 12.28 \times 10^3$$
$$\text{or } 1.228 \times 10^4$$
$$4.53 \times 10^3 + 1.271 \times 10^4 = 4.53 \times 10^3 + 12.71 \times 10^3$$
$$= 17.24 \times 10^3 = 1.724 \times 10^4$$

Numbers in the scientific notation can be divided or multiplied.

$$\frac{10^m}{10^n} = 10^{m-n}$$

$$\frac{6 \times 10^4}{2 \times 10^3} = \frac{6}{2} \times \frac{10^4}{10^3} = 3 \times 10^{4-3}$$

$$= 3 \times 10^1$$

$$10^m \times 10^n = 10^{m+n}$$

$$(2 \times 10^2) \times (3 \times 10^3) = 2 \times 3 \times 10^2 \times 10^3$$

$$= 2 \times 3 \times 10^{3+2}$$

$$= 6 \times 10^5$$

In division the main numbers are divided, while the power factors are subtracted. In multiplication the main numbers are multiplied, while the power factors are added.

Percentage

Per cent literally means "by the hundred," or parts per 100 parts. Per cent is the numerator of a fraction whose denominator is always 100.

$$\frac{50}{100} = 50\% \quad \frac{2.5}{100} = 2.5\%, \text{ etc.}$$

Common fraction, decimal fraction, and per cent are related terms. When using per cent, it is very important to recognize the *base*. If we have 8 grams of a material that contains 3 grams of sand, the ratio of the weight of the sand to the weight of the whole sample is 3 to 8. Here the base is the weight of the entire sample.

$$\frac{3}{8} = 0.375 = \frac{0.375}{1} = \frac{37.5}{100} = 37.5\%$$

Per cent	Common fraction	Decimal fraction
1.000	$\frac{1}{100}$	0.01
14.000	$\frac{14}{100}$	0.14
360.000	$\frac{360}{100}$	3.60
0.002	$\frac{2}{100{,}000}$	0.00002
0.0001	$\frac{1}{1{,}000{,}000}$	0.000001

Problems in Percentage

Problem: What weight of water is in a ton of sludge that is 60 per cent water? Sixty per cent means that 60/100 of any weight of sludge is water.

$$2000 \times \frac{60}{100} = 2000 \times 0.6 = 1200 \text{ lb.} \quad \text{Ans.}$$

Problem: The chlorine demand of a liquid is 9 ppm. After chlorination, the demand was 2 ppm. What is the per cent satisfaction of the demand?

$9 - 2 =$ part of demand satisfied.

$$\frac{9-2}{9} = \frac{7}{9} = 0.77$$

$0.77 \times 100 = 77\%$ satisfaction

Problem: What weight of water and alum are needed to make 5000 lb. of a 5 per cent solution?

$$5000 \times \frac{5}{100} = 250 \text{ lb. of alum}$$

$$5000 - 250 = 4750 \text{ lb. of water}$$

$$5000 \text{ lb. total}$$

Observe that a 5 per cent solution means 5 lb. of alum in 100 lb. of solution. Both the alum and the solution are expressed in the same units. This is, in general, the case when per cent (%) is being considered.

To arrive at the gallons of water required to make up the 5 per cent solution, it is necessary to divide pounds of water by the factor 8.34, which is pounds of water in each gallon.

$$\frac{4750}{8.34} = 570 \text{ gals. for all practical purposes}$$

Problem: If 3.75 lb. of H.T.H. containing 70 per cent available chlorine is dissolved in 30 gallons of water, what is the per cent (%) available chlorine in the solution?

$$3.75 \times 0.70 = \text{lbs. available chlorine} = 0.263 \text{ lbs.}$$

$$\frac{3.75 \times 0.70}{30 \times 8.34} = \frac{0.263}{250} = \text{lbs. available chlorine in 1 lb. of solu.} = .0105$$

$$100 \times 0.0105 = 1.05\ \%$$

In dilute solutions, the weight of water may be taken as equal to the weight of the solution.

Problem: Suspended solids in the influent is 600 ppm. and in the effluent 400 ppm. What is the per cent efficiency of removal?

$$600 - 400 = 200 \text{ removed}$$

$$\frac{200}{600} = 0.333$$

$$0.333 \times 100 = 33.3\%$$

Proportion

The indicated division of two pure numbers is a ratio. The ratio of 3 to 4 is usually expressed as 3:4 or sometimes as 3/4. A proportion is an equality of two ratios and is written in the form a:b = c:d or a/b = c/d and is expressed as "a" is to "b," as "c" is to "d." "a" and "d" are the *extremes*, while "b" and "c" are the *means*. The product of the extremes in a true proportion must always equal the product of the means.

Therefore, from the proportion—

$$a : b = c : d$$

we get

$$a \times d = c \times b$$

(extremes) (means)

To find any one of the values when the other three are known, proceed as follows:

$$(1) \qquad a = \frac{c \times b}{d}$$

$$(2) \qquad d = \frac{c \times b}{a}$$

$$(3) \qquad \frac{a \times d}{c} = b$$

$$(4) \qquad \frac{a \times d}{b} = c$$

The rule to remember is that in transferring the known value in the extremes or the means from one side of the equation to the other, when solving for the unknown value, it is necessary to divide the product of the means or extremes (as the case may be) by the value transferred.

Direct and Inverse Proportion

Two varying quantities are *directly proportional* if the ratio of corresponding values is constant.

Problem: If 3.75 lb. of H.T.H. is added to 30 gallons of water to make a solution of given strength, how many pounds will have to be added to 147 gallons to make a solution of the same concentration?

$$3.75 : 30 = x : 147$$

multiplying the extremes and the means gives

$$3.75 \times 147 = 30 \times x$$

and solving for x, the unknown portion, we have

$$\frac{3.75 \times 147}{30} = x = 18.37 \text{ lb.}$$

Two varying quantities are *inversely proportional* if their product is constant.

Problem: A gas holder contains 10,000 cubic feet of gas at a pressure of 3 pounds per square inch. If the volume is compressed to 3,000 cubic feet, what is the new pressure?

$$10{,}000 \times 3 = 3000 \times P$$

$$\frac{30{,}000}{3000} = 10 \text{ lb. per sq. in.}$$

Partitive Proportion

Partitive proportion divides a quantity into two or more parts that have specified ratios to each other.

Problem: Divide 27 into two parts that are in the ratio of 2:7.

$$2 + 7 = 9$$

$$27 \times \frac{2}{9} = 6 \quad \text{Ans.}$$

$$27 \times \frac{7}{9} = 21 \quad \text{Ans.}$$

Problem: Divide 81 into four parts that are in the ratios of 2:4:3:7.

$$2 + 4 + 3 + 7 = 16$$

$$\frac{2}{16} \times 81 = 10.12$$

$$\frac{4}{16} \times 81 = 20.25$$

$$\frac{3}{16} \times 81 = 15.20$$

$$\frac{7}{16} \times 81 = 35.44$$

Partitive proportion is useful in distributing a quantity among several others in proportion to the value of the latter.

Significant Figures

Two kinds of numbers are used in practical calculations. These are *absolutes and measurements*. An "absolute" expresses the value of something exactly. "Measurements" are the readings of scales, balances, meters, gages, and manometers. Every device for measuring has a threshold or sensitivity limit below which a change in the thing measured will not be detectable. *Every number representing a measurement is thus inexact or incomplete.*

If a meter shows a velocity of 3.21 ft./sec., we must recognize that it is really 3.21xyz and that all values for the decimal places beyond the second are real but unknown, using that particular meter. When measurements are combined by arithmetical operations, care is needed to avoid results that are absurd, meaningless, or actually fictitious.

A digit is one of the numbers 1, 2, 3, 4, 5, 6, 7, 8, 9, 0. The number 3427.67 means

$$3\,(1000) + 4\,(100) + 2\,(10) + 7\,(1) + 6\,(1/10) + 7\,(1/100)$$

The numbers within the parentheses are the unities in terms of which any number is expressed. The numbers before the parentheses are the significant figures. The number 2 is in the "tens" place, for example.

Determining Significant Numbers

A *significant* figure in a number expresses the number of *units* in the *place* in which it is located. The decimal point has nothing to do with significant figures. Zero may or may not be significant. In

0.006507, the zeros before the 6 are not significant, but the zero between 5 and 7 is significant.

The number of significant figures in a measurement is determined by the instrument and the conditions under which it is used. If the instrument is properly made, in good adjustment, and used by a trained observer, its readings will be correct *"as far as they go."* What does "as far as they go" mean? If we have a balance that will not indicate differences in mass of less than 0.1 gram, then any object weighed on this balance will be unknown beyond the first decimal place. If an object weighs somewhere between 12.8 and 12.9 and is recorded as 12.85, the weight of the object is really some absolute number between 12.8 and 12.9—i.e., 12.85 ± 0.05.

Maximum and Relative Errors

The 12.85 is a rounded-off value. It may be in error by 0.05 either way. We do not know the exact value but we do know that the *maximum error* is not greater than 0.05. The *maximum relative error* is the maximum possible error divided by the measurement and multiplied by 100 —in other words, expressed in per cent. In calculating the maximum relative error, only two significant figures are used in the measurement—the observed value and the maximum possible error. In this instance—

$$\frac{0.05}{12.85} \text{ is written } \frac{0.05}{13.00} \times 100 = 0.38\%$$

This is to say, the above weighing cannot be "trusted" to better than 0.38 per cent. The number of significant figures in a measurement gives some idea as to how far it can be trusted. When measurements are arithmetically combined, the errors in each of them go into the answer. The answer may become grossly misleading even if all operations in the pure arithmetic are strictly correct.

Significant Figures and Rounding Off

Unfortunately, as numbers are usually written, zero may have two meanings. It is used as a significant figure and also to locate the decimal point. In 22,400 as a measurement, we are not sure which meaning is to be given to the zeros. It would be better to write it as 2.24×10^4. This clearly indicates only three significant figures.

Problem: The following measurements are to be added: 22,400, 3642.2 and 23.24. The zeros in 22,400 are not significant.

```
  224xy.mn
  3642.2k
    23.24
  -------
```

Observe that xymnk are unknown.

Observe that the 4th vertical column containing x, 4, 2 is the first column, containing an unknown, that the eye reaches as it moves towards the right. All columns to the right of this x, 4, 2 column are to be discarded following the rules for rounding off. We now have

```
  224x-.      Assume x = 0 temporarily
  364-.
    2-.
  -------
  2606-.
```

This is now rounded back to 261--. or 26,100, with the last two zeros not significant.

Problem: Add 22.4, 3.675 and 0.8842.

```
  22.4---          round off and          22.4-
   3.675-          rewrite                 3.68
   0.8842                                  0.88
  -------                                 -----
                                          26.96
```

Since the second decimal place must be dropped, the answer is 27.0.

Problem: Multiply the following measurements together:

$$34.72 \times 359.74 \times 0.0016$$

If we multiply as in pure arithmetic, we get 19.98267648. The operator vaguely recognizes that all the decimal places obtained are not necessary and proceeds probably to record the result as 19.98, but without knowing why he kept two decimal places instead of one or three. Better procedure would be as follows: Note the factor with the smallest number of significant figures. This is 0.0016 with only two significant figures. Round off the other factors so that they have only one more significant figure. We now have

$$34.7 \times 360 \times 0.0016$$
$$34.7 \times 360 = 12492 \text{ rounded off to } 12500$$
$$12500 \times 0.0016 = 20.0000 \text{ rounded off to } 20$$

The answer in general must have no more significant figures than the factor with the smallest number of significant figures.

Rules Governing Calculation of Errors

The following rules govern all cases:

Maximum error of a sum = sum of maximum errors of measurements added.

Maximum error of a difference = sum of maximum errors of measurements subtracted.

% error of product = sum of % errors of factors.

% errors of quotient = sum of % errors of factors.

Problem: Divide 2.047 by 1.24. Calculate the % maximum relative error in the answer.

$$\frac{2.047}{1.24} = 1.65$$

Max. rel. error in $2.047 = 0.025\%$
Max. rel. error in $1.24 = 0.42\%$
Max. rel. error in answer $= 0.44\%$

Physical Units

A *unit* is a single thing isolated by the mind and called one. A *number* is a unit or a collection of units secured by the processes of addition, subtraction, multiplication, and division. An *abstract* or *pure* number is used without reference to any particular thing. A *concrete number applies to a particular thing and is the product of an abstract number and a physical unit.*

Examples of physical units are—1 foot; 1 acre; 1 gallon; etc.

Rule: Concrete numbers can be added or subtracted only when they possess the same physical unit.

Rule: Concrete numbers having the same or different physical units can be multiplied, and, when this multiplication takes place, the physical units must also be multiplied, thereby creating a new physical unit.

$$6 \text{ ft.} \times 7 \text{ ft.} = 6 \text{ (1 ft.)} \times 7 \text{ (1 ft.)}$$
$$= 6 \times 7 \times (1 \text{ ft.}) \times (1 \text{ ft.})$$
$$= 42 \times \text{ft.} \times \text{ft.}$$
$$= 42 \text{ ft.}^2$$

1 ft.^{2} or 1 sq. ft. is a new physical unit
Similarly 4 (1 ton) x 3 (1 mile) =
12 (1 ton) (1 mile) = 12 ton-miles

A ton-mile is a definite physical unit and represents something done, associated with carrying a ton through a distance of 1 mile.

5 (1 man) x 4 (1 hour) = 20 (man-hours)
1000 (1 person) x 365 (1 day) = 365,000 capita-days

Rule: When concrete numbers are divided, the physical units are also divided. A physical unit in the numerator cancels a like physical unit in the denominator.

In general when the physical units are divided a new physical unit is created.

$$\frac{20 \text{ miles}}{2 \text{ hours}} = \frac{20}{2}\left(\frac{1 \text{ mile}}{1 \text{ hour}}\right) = 2\left(\frac{1 \text{ mile}}{1 \text{ hour}}\right)$$

or 2 *miles per hour*

Important Uses of Physical Units

The result of every calculation with concrete numbers will in general be associated with a physical unit that gives meaning to the numerical answer. The numerical answer should always be accompanied by its physical unit. Columns of data must always have the physical units expressed at the top. An important use for physical units is to determine how figures are to be combined to give the desired answer and to check the correctness of formulas for omitted quantities, and incorrect units.

Both sides of an equation must reduce to the same physical unit.

Problems: A flow is 40 ft.^{3}/sec. and the cross-sectional area is 20 ft.^{2} What is the linear velocity of flow?

$$\frac{\text{ft.}^3\text{/sec.}}{\text{ft.}^2} = \frac{\text{ft.}^3}{\text{sec. x ft.}^2} = \text{ft./sec.}$$

$$\frac{40 \text{ ft.}^3\text{/sec.}}{20 \text{ ft.}^2} = 2 \text{ ft./sec.}$$

Observe that physical units can be carried through a calculation without using the numerical values. This is done to determine the physical unit of the answer.

Rates

The concept of a rate is very important in all practical affairs. Rates consist of a number multiplied by a physical unit. This physical unit is formed by dividing one physical unit by another and is called the rate unit.

Example: A channel delivers in steady flow 5500 cubic feet (ft.^{3}) in 5 hours.

$$\frac{5500 \text{ ft.}^3}{5 \text{ hours}} = \frac{5500}{5}\left(\frac{\text{ft.}^3}{\text{hr.}}\right) = 1100 \text{ ft.}^3\text{/hr.}$$

Here we have a rate formed by dividing two concrete numbers. The rate should be visualized as the product of 1100 by the rate unit, which is 1 ft.^{3}/hr. This latter is read one cubic foot per hour, in which the "per" is equivalent to "each."

To develop facility in setting up and using rates, some thought must be given to the physical meaning of division. Division means distribution. The totality of things represented by the numerator are equally distributed among the units of the denominator. The result of the division is the parts of the numerator per, or to each part of the denominator.

$$\frac{50 \text{ ft.}}{2 \text{ sec.}} = 25\left(\frac{\text{ft.}}{\text{sec.}}\right) = 25 \text{ ft. per sec.}$$

$$\frac{500 \text{ gals.}}{2 \text{ persons}} = 250\left(\frac{\text{gal.}}{\text{person}}\right) = 250 \text{ gals. per person}$$

$$\frac{50 \text{ mg.}}{25 \text{ liters}} = 2\left(\frac{\text{mg.}}{\text{liter}}\right) = 2 \text{ mg. per liter}$$

$$\frac{75 \text{ cents}}{5 \text{ oranges}} = 15\left(\frac{\text{cents}}{\text{orange}}\right) = 15\text{c per orange}$$

$$\frac{100 \text{ cu. ft.}}{5 \text{ men} \times 4 \text{ hrs.}} = 5\left(\frac{\text{ft.}^3}{\text{men} \times \text{hr.}}\right) = 5 \text{ cu. ft. per man per hr.}$$

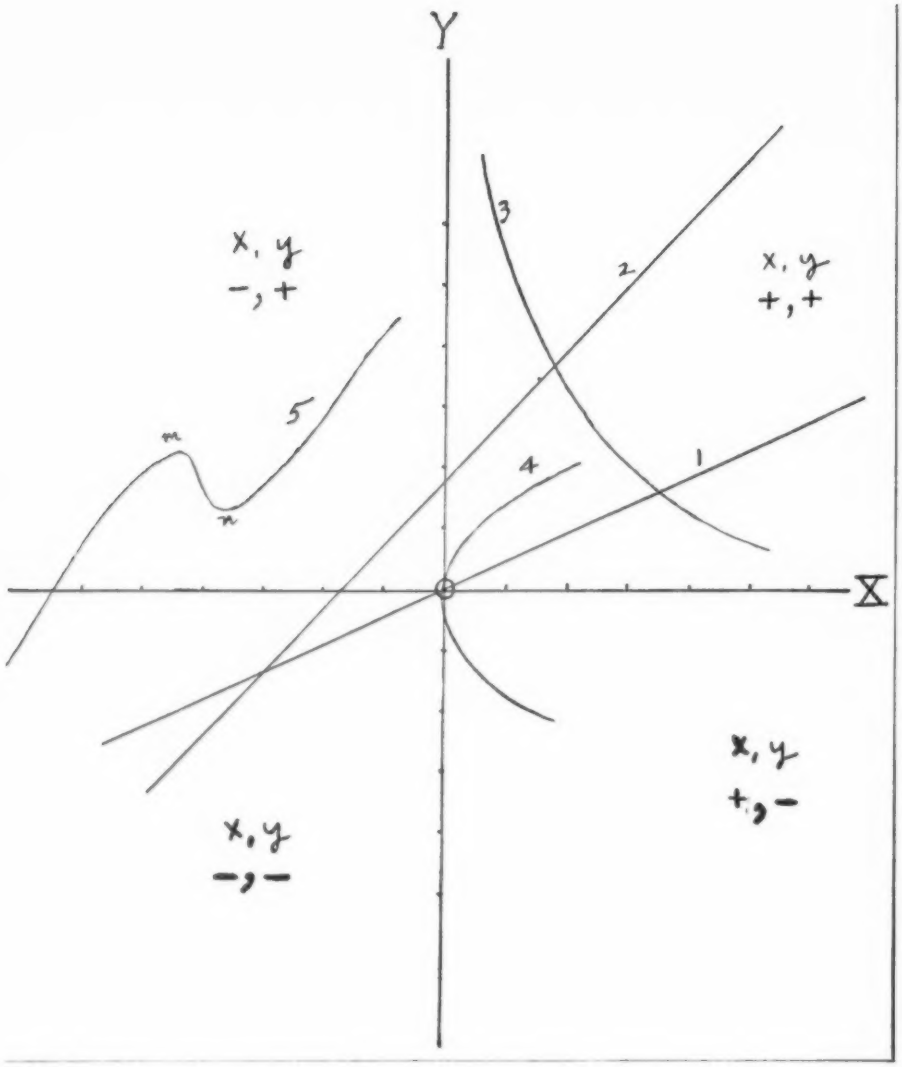


Fig. 1. Illustration of Types of Graphs

Equivalents and Conversions

The following list of equivalents cover all ordinary needs in sewage and water problems:

1 meter	=	100 centimeters
1 centimeter	=	10 millimeters
1 inch	=	2.54 centimeters
1 pound	=	453.6 grams
1 gram	=	15.43 grains
1 ounce (av.)	=	28.3 grams
1 cubic foot	=	7.48 gallons
1 gallon of water	=	8.34 pounds
1 cu. ft. of water	=	62.4 pounds
1 cubic foot	=	28.3 liters
1 gram per liter	=	1 ounce per cu. ft.
1 gram per milliliter	=	62.4 pounds per cu. ft.
1 milligram per liter	=	1 part per million
1 grain per gallon	=	17.1 parts per million
1 part per million	=	8.34 pounds per million gallons
1 acre	=	43,560 square feet
1 gallon	=	3.785 liters
1 liter	=	1000 milliliters

The above list of equivalents can be extended indefinitely.

A much more comprehensive table of equivalents is to be found in the 1945 Reference and Data Number of *Water Works & Sewerage.*

Calculating Units

Sometimes a desired unit is not in the given list. If the desired unit is not present, it usually may be calculated with little difficulty.

The calculation involves two principles as follows: (a) physical units may be cancelled; (b) any quantity divided by its equivalent equals unity.

For example, to obtain the number of centimeters in one foot, this calculation resolves itself simply into multiplying the number of centimeters in one inch by the number of inches in one foot.

$$12 \times 2.54 = 30.48 \text{ cm.}$$

Problem: What quantity of water weighs one pound. If proportion, explained earlier, is used to convert the value of 8.34 lbs. per gallon shown in the table to gallons per pound we have

$$8.34 \text{ lbs.} : 1 \text{ gal.} = 1 \text{ lb.} : x \text{ gals.}$$
$$8.34\,x = 1 \times 1$$
$$x = 0.12 \text{ gals.}$$
(Not quite a pint)

Converting One Set of Units to Another

Problem: One ounce per gallon is equal (a) to how many pounds per cubic foot and (b) to how many grams per liter?

$$1\frac{\text{ounce}}{\text{gallon}} = 1\frac{\text{ounce}}{\text{gallon}} \times \frac{7.48 \text{ gallon}}{1 \text{ cu. ft.}} \times \frac{1 \text{ pound}}{16 \text{ ounce}}$$
$$= \frac{7.48 \text{ pound}}{16 \text{ cu. ft.}} = 0.468\frac{\text{pound}}{\text{cu. ft.}}$$

$$1\frac{\text{ounce}}{\text{gallon}} = 1\frac{\text{ounce}}{\text{gallon}} \times \frac{1 \text{ gallon}}{3.785 \text{ liter}} \times \frac{28.3 \text{ grams}}{1 \text{ ounce}}$$
$$= \frac{28.3 \text{ grams}}{3.785 \text{ liter}} = 7.48\frac{\text{grams}}{\text{liter}}$$

Numerical values for equivalents or conversion factors in one set of units may be converted to another set of units in the above very simple manner.

Variation, Graphs, Equations

The various meters in a water or sewage treatment plant change their readings with changing conditions. Those quantities whose values may change are called variables. Constants do not change their values in specific problems.

If two variables are connected in some way so that a change in one produces a change in the other, we say that one is a function of the other. This concept is very important. In studying any happening scientifically, it is necessary to locate all variables, changes in which affect the happening under study. If possible, two variables are selected for study and all others kept constant. One of the two selected variables is arbitrarily changed and the effect upon the other is measured. The one changed arbitrarily is called the independent variable and the other the dependent variable.

The note-book now contains a table of data in two columns. For each "setting" of the independent variable, there will be a value for the dependent variable, the two values constituting a corresponding pair.

Graphs

In order to facilitate the study of the data, a graph is plotted. Ordinary graph paper shows a grid or net formed by two sets of parallel lines at right angles to each other.

In plotting, the *origin* is selected first. If the data includes negative values, the origin must be moved toward the center of the sheet. If negative values are absent, the lower left corner is selected. Two accentuated lines in the net at right angles to each other and both passing through the origin are designated as the *axes.*

Scales are assigned to the axes such that the smallest square has its sides represent 1, 10, or 100 depending on the range of the data. Scales need not be the same for both axes but they should be selected such that the graph roughly extends diagonally across the entire sheet. The scales must be such that the data can be plotted easily and quickly. A point is plotted by locating the value of the independent variable on the horizontal scale and from this latter point measuring on a vertical line, a distance equal to the dependent variable on its own scale. Positive values are measured to the right and upward and negative values to the left and downward. Usually a smooth curve is drawn or "averaged" through the points. This curve is the graph. Each point on the graph represents a pair of corresponding values for the variables. Several types of graphs are illustrated in Fig. 1.

Graphs are important because (a) the mind can note by a glance the general nature of the relationship between two variables; (b) a graph facilitates interpolation which means reading from the graph pairs of corresponding values that were not measured; (c) a graph shows up peculiarities; (d) graphs are used very frequently for rapid calculation; (e) graphs are used for computation that could not be made in any other way.

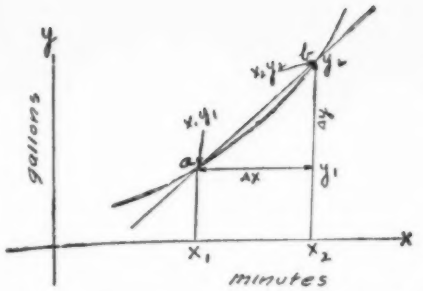


Fig. 2. Graph Showing Rate of Flow, Not Constant

Algebraic Equations

So far, we have represented our data in two ways: (a) a table of measured values; (b) a graph. There is another way to represent a function. This is by means of an algebraic equation.

Consider Curve 1 in Fig. 1. This graph shows a very common and important type of relationship between two variables. The graph is a straight line passing through the origin. Whenever this occurs, the two variables are *directly proportional*. If the dependent variable be called 'y' and the independent variable 'x,' we say in this case that 'y' varies directly with 'x' or that 'y' is proportional to 'x' or

$$\frac{x_1}{y_1} = \frac{x_2}{y_2}$$

where the subscripts denote corresponding pairs.

Since the ratio here of any 'x' to its corresponding 'y' is a constant we can write

$$\frac{x_1}{y_1} = \text{some constant and}$$

therefore

$$y = kx$$

where k is the proportionality constant.

Note: If we know the value of k, we can dispense with the graph. $y = kx$ is a simple example of an algebraic equation that represents both a table of data and a graph. $y = kx$ is called the equation of the curve.

Consider Curve 2 in Fig. 1. Here the plotted measurements give a straight line but the two variables are not directly proportional. The equation for this graph is $y = ax + b$ where 'a' and 'b' are constants. 'a' is the slope of the graph and 'b' is the distance from the origin to the curve as measured along the 'y' axis.

Consider Curve 3. The product of 'x' and 'y' is constant, or $xy = k$. This is a common type of variation. We say that 'x' and 'y' are *inversely proportional*. This is so whenever the product of two variables is constant. An application of this is seen in the Venturi meter where the area times the velocity up-stream equals the area times the velocity in the throat.

Slopes, Rates, Areas Under Curves

After data from a table have been plotted, some useful computations can be made from the graph. It is often necessary to know how rapidly a varying quantity is increasing or decreasing, i.e., its rate of change. This is the amount of change in the dependent variable per unit change in the independent variable.

Rates

On a graph, the rate of change is shown by the steepness of the curve and not by the height above the 'x' axis at any point. The steepness of any curve is expressed by giving its slope. Slope and rate of change are synonymous. Two kinds of rates must be considered: (a) average rate over an interval; (b) instantaneous rate at a point.

In Fig. 2 the curve represents total gallons that have flowed as a function of the time in minutes. The graph shows that the rate of flow is not constant because rates of change are constant only on a straight line graph. As a point travels from 'a' to 'b' along the curve, its coordinates change from x_1, y_1 to x_2, y_2. The increase in the gallons from 'a' to 'b' is $y_2 - y_1$ and the increase in the minutes is $x_2 - x_1$.

$$\frac{\text{increase in y}}{\text{increase in x}} = \text{average rate of change over the interval} = \text{change in 'y' per unit change in 'x'}$$

Note that the average rate of change over an interval tells nothing about how the graph changes throughout the interval. It is very important to distinguish between *average rate of change over an interval* and *the average value of the varying quantity* over the same interval.

In Fig. 3 an important idea is illustrated. If a point on the graph within an interval

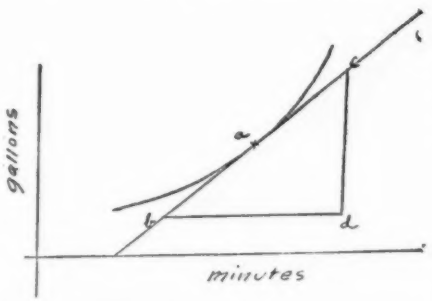


Fig. 3. Determining Slope or Instantaneous Rate at a Point

is kept fixed while the interval is steadily shortened, the average rate of change will approach and become equal to the instantaneous rate. The instantaneous rate or slope at a point is determined by drawing a tangent to the graph at the point in question and determining the average rate for the tangent. The average rate for the tangent is determined by drawing any right triangle bdc on the tangent and dividing dc by bd with the sides of the triangle expressed in the scale units.

Area Under a Curve

In Fig. 4 another important quantity for many graphs is illustrated. This is the area under the curve and bounded by the 'x' axis and the ordinates at the beginning and the end of an interval. This area frequently has physical significance. It is approximated by splitting the interval into rectangles and drawing the top of each rectangle across the curve at a point that makes the area of triangle 1 equal to triangle 2. The height of each rectangle is measured in scale units and multiplied by the width in scale units. The area under the curve is the sum of the areas of these rectangles. The area may also be determined by a planimeter.

One very important use of this area is to obtain the average value of a varying

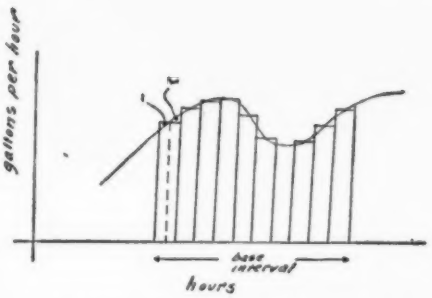


Fig. 4. Determining Area Under a Curve

quantity over an interval. This latter is obtained by dividing the total area by the base interval. This average value will be based on all the values of the varying quantity.

Average values for varying quantities determined in this way are usually more accurate than when based upon a limited number of readings. Whenever the product of two variables is sought, it can be obtained by plotting one variable as the function of the other and measuring the area under the curve between definite limits. If the velocity of flow is plotted as a function of time, the area under the curve represents total quantity. The volume of a reservoir or irregular object can be determined by plotting the cross-sectional area as a function of the height. If the concentration of suspended solids in a tank is non-uniform, the amount per cubic foot plotted as a function of the depth will give a curve, the area under which will be the amount of suspended solids in a vertical column 1 square foot in section. Accuracy increases with the number of levels sampled.

Algebraic Signs and Symbols

An algebraic symbol is a letter representing a definite object or thing. The idea of quantity is always present. Algebraic signs indicate quality, aggregation, or operation.

Addition $a + b$

Subtraction $a - b$

Multiplication $a \times b$ or $a \cdot b$ or ab

Division a/b or $a \div b$ or $\frac{a}{b}$

Powers $a \times a = a^2$

$a \times a \times a \times \ldots$ n times $= a^n$

Square root $\sqrt{a} = a^{\frac{1}{2}}$

nth root $\sqrt[n]{a} = a^{\frac{1}{n}}$

Fractional exponent—

$$a^{\frac{m}{n}} = (a^{\frac{1}{n}})^m = (a^m)^{\frac{1}{n}} = \sqrt[n]{a^m}$$

Meaning of Signs

The signs (+) and (—) have two distinct meanings in algebra. They are signs of operation for addition and subtraction. For addition or subtraction, they are read "plus" and "minus." They are also signs of quality and are read "positive" and "negative." Positive and negative mean "opposite." One must be given a meaning, whereupon the other becomes the opposite. If distances upward are arbitrarily marked positive, distances downward are negative. Credit may be marked positive and indebtedness negative. Pressure may be positive and vacuum negative. Temperatures above zero are positive and below negative.

Examples of Algebraic Operations

$$(+a) + (+b) \equiv a + b$$
$$(-a) + (+b) \equiv -a + b \equiv b - a$$
$$(-a) + (-b) \equiv -a - b \equiv -(a + b)$$
$$(+a)(+b) \equiv ab$$
$$(-a)(+b) \equiv -ab$$
$$(-a)(-b) \equiv +ab$$
$$n(a + b) \equiv na + nb$$
$$-n(a + b) \equiv -na - nb$$
$$-n(a - b) \equiv -na + nb$$
$$\frac{-a}{+b} = -\frac{a}{b}$$
$$\frac{-a}{-b} = \frac{a}{b}$$

Signs of Aggregation

The signs of aggregation usually used are parentheses ( ), brackets [ ], braces { }, and vinculum —. An expression within a pair of signs of aggregation must be treated as a unit. To remove signs of aggregation, start with the innermost pair and work outwards.

$$\{\ 6 - [2 - (8 - 2)]\ \} = 10$$

Rules for Exponents

$$a^m \times a^n = a^{m+n}$$
$$\frac{a^m}{a^n} = a^{m-n}$$
$$(a^m)^n = a^{mxn}$$
$$(a^m)^{\frac{1}{n}} = \sqrt[n]{a^m}$$
$$a^{-m} = \frac{1}{a^m}$$

Manipulation of a Formula

In using a formula for calculation, rearrangement is frequently necessary to isolate the variable whose value is to be computed. Given the following formula, isolate x.

$$\sqrt{k} = \frac{x^2y + 5z}{3} + 6$$

Principles Governing Rearrangements

The following principles govern rearrangements:

(a) Equals added to or subtracted from both sides of an equation do not alter the equality.

(b) Equality is not altered if both sides of an equation are divided by the same number.

(c) Equality is not altered if both sides are multiplied by the same number.

(d) Equality is not altered if equal powers and roots are applied to both sides.

If —6 is added to both sides above,

$$\sqrt{k} - 6 = \frac{x^2y + 5z}{3}$$

Multiply both sides by 3

$$3\sqrt{k} - 18 = x^2y + 5z$$

Add —5z to both sides

$$3\sqrt{k} - 18 - 5z = x^2y$$

Divide both sides by y

$$\frac{3\sqrt{k} - 18 - 5z}{y} = x^2$$

Take square root of both sides

$$\sqrt{\frac{3\sqrt{k} - 18 - 5z}{y}} = x$$

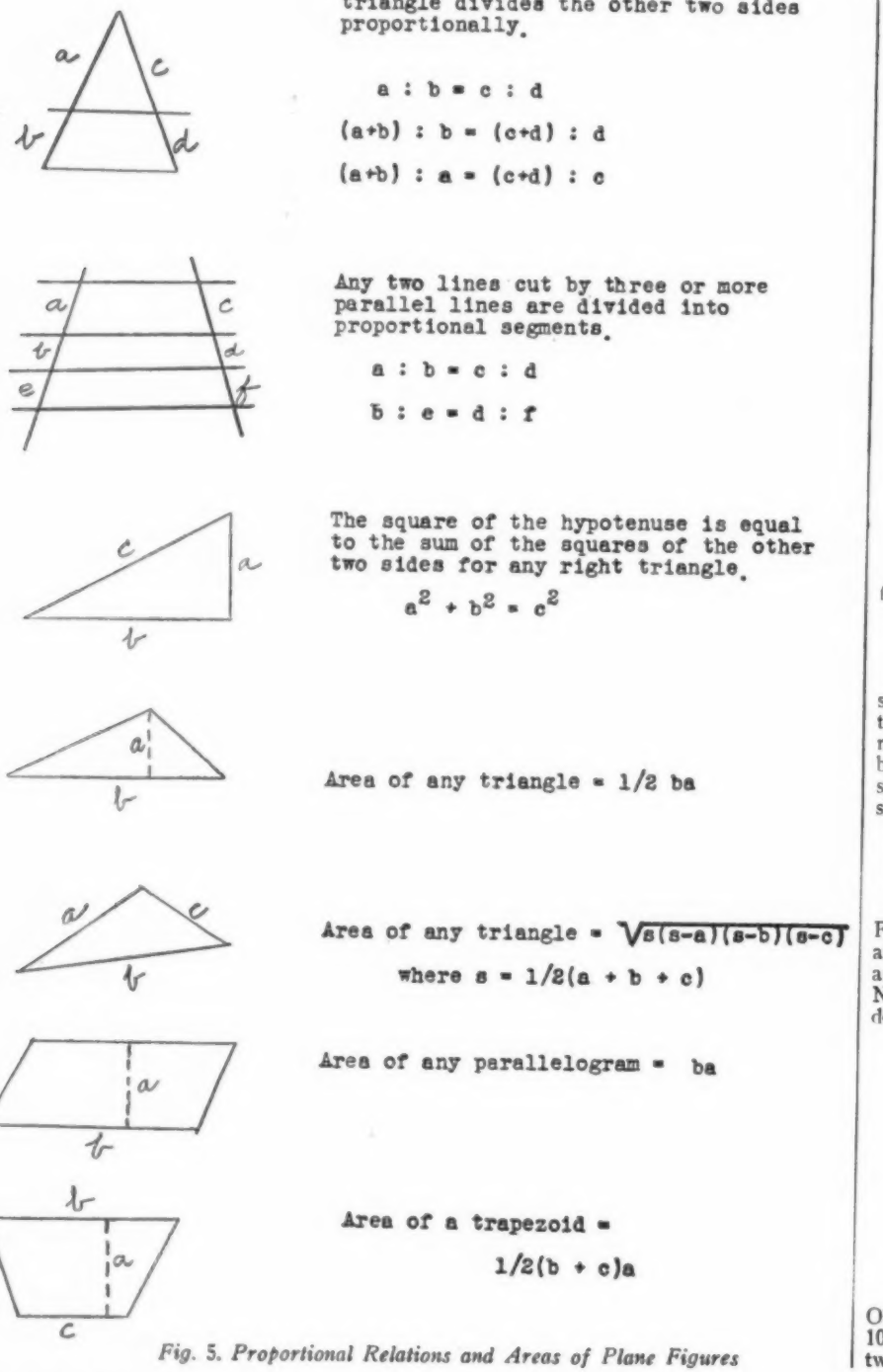


Fig. 5. Proportional Relations and Areas of Plane Figures

Areas and Volumes

Areas and volumes of various geometric figures are shown in Figs. 5 and 6.

Logarithms

Logarithms are exponents. Their use saves time and labor in extended calculation involving only multiplication, division, roots, and powers. Some calculations can be made in no other way. The understanding of pH requires their use. Consider.

$N = a^x$ where
N is any number
a is a constant called the base
x is a variable exponent that changes when N changes

For common logarithms, the base is 10 and hence, $N = 10^x$ where x is the logarithm to the base 10 and is written $\log N = x$. The logarithmic system is evident from the following tabulation:

$1 = 10^0$
$10 = 10^1$
$100 = 10^2$
$1000 = 10^3$
$1{,}000{,}000 = 10^6$
$0.1 = 10^{-1}$
$0.01 = 10^{-2}$
$0.001 = 10^{-3}$
$\log 1 = 0$
$\log 10 = 1$
$\log 100 = 2$
$\log 1000 = 3$
$\log 1{,}000{,}000 = 6$
$\log 0.1 = -1 = 9.0000 - 10$
$\log 0.01 = -2 = 8.0000 - 10$
$\log 0.001 = -3$

Observe that since the $\log 1 = 0$ and $\log 10 = 1$, the logarithms of all numbers between 1 and 10 are numbers between 0 and

1. For example $3 = 10^{0.4771}$ or $\log 3 = 0.4771$.

Any number can be written as a number between 1 and 10 times some power of 10. Observe the following:

$$137 = 1.37 \times 10^{2}$$
$$22740 = 2.2740 \times 10^{4}$$
$$0.0247 = 2.47 \times 10^{-2}$$
$$0.0004735 = 4.735 \times 10^{-4}$$

The power of 10 is equal to the number of decimal places thru which the point has been shifted, positive if moved to the left and negative if moved to the right. In Fig. 7 the table of logarithms shown is a four place table. All numbers with more than four significant figures should be rounded off to four figures before entering the table. A four place table is sufficient for most purposes.

How to Find a Logarithm of a Number

Find the log of 467.8 in the table. Assume that the table gives the logs of numbers from 1 to 10 only. Mentally shift the decimal point in 467.8 to 4.678×10^{2}. Enter the table with 46 in the extreme left column and proceed horizontally until the column headed by the third figure of the number is reached. Result up to this point is 6693. Again proceed horizontally until the column in the table of Proportional Parts, headed by the fourth figure in the number is reached. Here we find 7. Add this 7 to 6693 and we have 6700 or 0.6700. This 0.6700 is the log of 4.678. With this we must combine the log 10^{2}. The log 10^{2} is 2. When numbers are multiplied, their logs are added, hence 0.6700 + 2 or 2.6700 is the log of 467.8.

Find the log of 0.007284 in the table. Shift the decimal point so that the number becomes 7.284×10^{-3} and enter the table with 7284.

$$\log 7.284 = 0.8623$$
$$\log 10^{-3} = -3$$
$$\log 0.007284 = 0.8623 - 3$$

Important: Logarithms written in the form of 0.8623—3 may be changed in form but not in value by adding the same quantity to both the 0.8623 and the 3. The 0.8623—3 can be written as 7.8623—10 or as 5.8623—8. For logarithms of numbers less than one it is important to recognize that (a—b) = (a+n) — (b+n).

How to Find a Number, Given the Logarithm

Find the number whose logarithm is 1.8423. Mentally change this to 0.8423+1. Enter the body of the table and locate 8420 which is next lower than the given 8423. Opposite the 8420 is 69 and above the 8420 is column heading 5. Our number so far is 6.95. Next, 8420 is 3 less than 8423. Go horizontally from 8420 until 3 is reached in the table of Proportional Parts. This 3 is in the column headed by 5. Our number so far is 6.995 determined by the 0.8423 part of the logarithm. The +1 means that the 6.955 is to be multiplied by 10^{1} or our final number is 69.55.

Find the number whose logarithm is 0.5842—2. From the tables the number whose log is 0.5842 is 3.839. The —2 means that 3.839 is to be multiplied by 10^{-2} or our final number is 0.03839.

Problem: Calculate the following

$$\sqrt[3]{\frac{4.230^{2} \times 153.7}{0.001972}}$$

log answer = 2.0481
answer = 111.7

Rules used for the above example are:

$$\log a^{n} = n \log a$$
$$\log ab = \log a + \log b$$
$$\log \frac{a}{b} = \log a - \log b$$
$$\log \sqrt[n]{a} = \frac{\log a}{n}$$

Density and Specific Gravity

THE density of any object is the ratio of its mass to its volume. The mass is taken as equal to its weight.

$$D = \frac{W}{V}$$

The numerical value of the density depends upon the units of weight and volume. Three sets of units are common. They

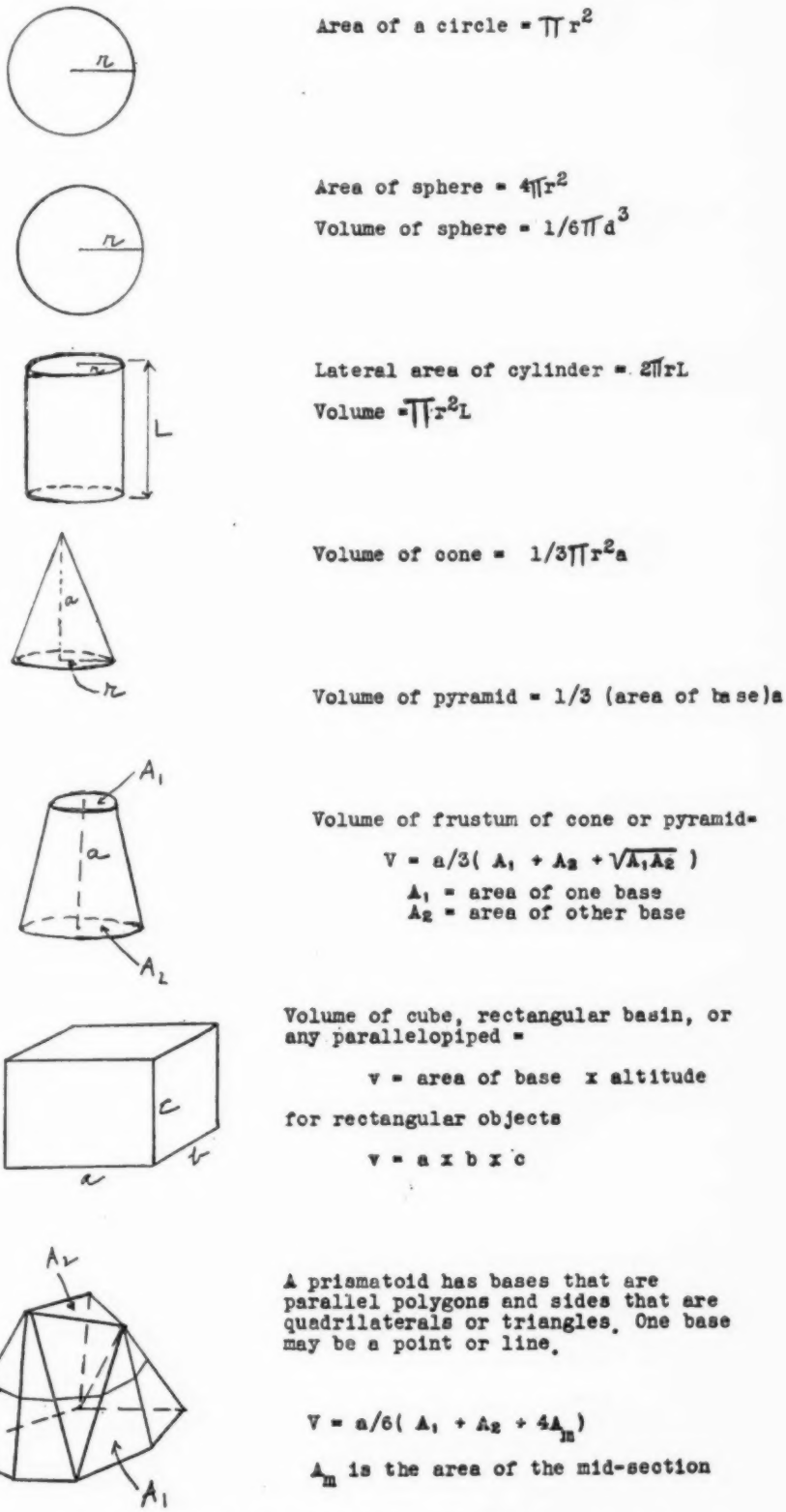

r
Area of a circle = ∏r²
r
Area of sphere = 4∏r²
Volume of sphere = 1/6∏d³
r
L
Lateral area of cylinder = 2∏rL
Volume = ∏r²L
a
r
Volume of cone = 1/3∏r²a
Volume of pyramid = 1/3 (area of base)a
A₁
a
A₂
Volume of frustum of cone or pyramid=
V = a/3(A₁ + A₂ + √A₁A₂)
A₁ = area of one base
A₂ = area of other base
c
b
a
Volume of cube, rectangular basin, or any parallelopiped =
v = area of base x altitude
for rectangular objects
v = a x b x c
A₂
A₁
A prismatoid has bases that are parallel polygons and sides that are quadrilaterals or triangles. One base may be a point or line.
V = a/6(A₁ + A₂ + 4Aₘ)
Aₘ is the area of the mid-section

Fig. 6. Areas and Volumes of Plane and Solid Geometrical Figures

Natural Numbers	0	1	2	3	4	5	6	7	8	9	Proportional Parts 1 2 3	4 5 6	7 8 9
10	0000	0043	0086	0128	0170	0212	0253	0294	0334	0374	4 8 12	17 21 25	29 33 37
11	0414	0453	0492	0531	0569	0607	0645	0682	0719	0755	4 8 11	15 19 23	26 30 34
12	0792	0828	0864	0899	0934	0969	1004	1038	1072	1106	3 7 10	14 17 21	24 28 31
13	1139	1173	1206	1239	1271	1303	1335	1367	1399	1430	3 6 10	13 16 19	23 26 29
14	1461	1492	1523	1553	1584	1614	1644	1673	1703	1732	3 6 9	12 15 18	21 24 27
15	1761	1790	1818	1847	1875	1903	1931	1959	1987	2014	3 6 8	11 14 17	20 22 25
16	2041	2068	2095	2122	2148	2175	2201	2227	2253	2279	3 5 8	11 13 16	18 21 24
17	2304	2330	2355	2380	2405	2430	2455	2480	2504	2529	2 5 7	10 12 15	17 20 22
18	2553	2577	2601	2625	2648	2672	2695	2718	2742	2765	2 5 7	9 12 14	16 19 21
19	2788	2810	2833	2856	2878	2900	2923	2945	2967	2989	2 4 7	9 11 13	16 18 20
20	3010	3032	3054	3075	3096	3118	3139	3160	3181	3201	2 4 6	8 11 13	15 17 19
21	3222	3243	3263	3284	3304	3324	3345	3365	3385	3404	2 4 6	8 10 12	14 16 18
22	3424	3444	3464	3483	3502	3522	3541	3560	3579	3598	2 4 6	8 10 12	14 15 17
23	3617	3636	3655	3674	3692	3711	3729	3747	3766	3784	2 4 6	7 9 11	13 15 17
24	3802	3820	3838	3856	3874	3892	3909	3927	3945	3962	2 4 5	7 9 11	12 14 16
25	3979	3997	4014	4031	4048	4065	4082	4099	4116	4133	2 3 5	7 9 10	12 14 15
26	4150	4166	4183	4200	4216	4232	4249	4265	4281	4298	2 3 5	7 8 10	11 13 15
27	4314	4330	4346	4362	4378	4393	4409	4425	4440	4456	2 3 5	6 8 9	11 13 14
28	4472	4487	4502	4518	4533	4548	4564	4579	4594	4609	2 3 5	6 8 9	11 12 14
29	4624	4639	4654	4669	4683	4698	4713	4728	4742	4757	1 3 4	6 7 9	10 12 13
30	4771	4786	4800	4814	4829	4843	4857	4871	4886	4900	1 3 4	6 7 9	10 11 13
31	4914	4928	4942	4955	4969	4983	4997	5011	5024	5038	1 3 4	6 7 8	10 11 12
32	5051	5065	5079	5092	5105	5119	5132	5145	5159	5172	1 3 4	5 7 8	9 11 12
33	5185	5198	5211	5224	5237	5250	5263	5276	5289	5302	1 3 4	5 6 8	9 10 12
34	5315	5328	5340	5353	5366	5378	5391	5403	5416	5428	1 3 4	5 6 8	9 10 11
35	5441	5453	5465	5478	5490	5502	5514	5527	5539	5551	1 2 4	5 6 7	9 10 11
36	5563	5575	5587	5599	5611	5623	5635	5647	5658	5670	1 2 4	5 6 7	8 10 11
37	5682	5694	5705	5717	5729	5740	5752	5763	5775	5786	1 2 3	5 6 7	8 9 10
38	5798	5809	5821	5832	5843	5855	5866	5877	5888	5899	1 2 3	5 6 7	8 9 10
39	5911	5922	5933	5944	5955	5966	5977	5988	5999	6010	1 2 3	4 5 7	8 9 10
40	6021	6031	6042	6053	6064	6075	6085	6096	6107	6117	1 2 3	4 5 6	8 9 10
41	6128	6138	6149	6160	6170	6180	6191	6201	6212	6222	1 2 3	4 5 6	7 8 9
42	6232	6243	6253	6263	6274	6284	6294	6304	6314	6325	1 2 3	4 5 6	7 8 9
43	6335	6345	6355	6365	6375	6385	6395	6405	6415	6425	1 2 3	4 5 6	7 8 9
44	6435	6444	6454	6464	6474	6484	6493	6503	6513	6522	1 2 3	4 5 6	7 8 9
45	6532	6542	6551	6561	6571	6580	6590	6599	6609	6618	1 2 3	4 5 6	7 8 9
46	6628	6637	6646	6656	6665	6675	6684	6693	6702	6712	1 2 3	4 5 6	7 7 8
47	6721	6730	6739	6749	6758	6767	6776	6785	6794	6803	1 2 3	4 5 5	6 7 8
48	6812	6821	6830	6839	6848	6857	6866	6875	6884	6893	1 2 3	4 4 5	6 7 8
49	6902	6911	6920	6928	6937	6946	6955	6964	6972	6981	1 2 3	4 4 5	6 7 8
50	6990	6998	7007	7016	7024	7033	7042	7050	7059	7067	1 2 3	3 4 5	6 7 8
51	7076	7084	7093	7101	7110	7118	7126	7135	7143	7152	1 2 3	3 4 5	6 7 8
52	7160	7168	7177	7185	7193	7202	7210	7218	7226	7235	1 2 2	3 4 5	6 7 7
53	7243	7251	7259	7267	7275	7284	7292	7300	7308	7316	1 2 2	3 4 5	6 6 7
54	7324	7332	7340	7348	7356	7364	7372	7380	7388	7396	1 2 2	3 4 5	6 6 7

Natural Numbers	0	1	2	3	4	5	6	7	8	9	Proportional Parts 1 2 3	4 5 6	7 8 9
55	7404	7412	7419	7427	7435	7443	7451	7459	7466	7474	1 2 2	3 4 5	5 6 7
56	7482	7490	7497	7505	7513	7520	7528	7536	7543	7551	1 2 2	3 4 5	5 6 7
57	7559	7566	7574	7582	7589	7597	7604	7612	7619	7627	1 2 2	3 4 5	5 6 7
58	7634	7642	7649	7657	7664	7672	7679	7686	7694	7701	1 1 2	3 4 4	5 6 7
59	7709	7716	7723	7731	7738	7745	7752	7760	7767	7774	1 1 2	3 4 4	5 6 7
60	7782	7789	7796	7803	7810	7818	7825	7832	7839	7846	1 1 2	3 4 4	5 6 6
61	7853	7860	7868	7875	7882	7889	7896	7903	7910	7917	1 1 2	3 4 4	5 6 6
62	7924	7931	7938	7945	7952	7959	7966	7973	7980	7987	1 1 2	3 3 4	5 6 6
63	7993	8000	8007	8014	8021	8028	8035	8041	8048	8055	1 1 2	3 3 4	5 5 6
64	8062	8069	8075	8082	8089	8096	8102	8109	8116	8122	1 1 2	3 3 4	5 5 6
65	8129	8136	8142	8149	8156	8162	8169	8176	8182	8189	1 1 2	3 3 4	5 5 6
66	8195	8202	8209	8215	8222	8228	8235	8241	8248	8254	1 1 2	3 3 4	5 5 6
67	8261	8267	8274	8280	8287	8293	8299	8306	8312	8319	1 1 2	3 3 4	5 5 6
68	8325	8331	8338	8344	8351	8357	8363	8370	8376	8382	1 1 2	3 3 4	4 5 6
69	8388	8395	8401	8407	8414	8420	8426	8432	8439	8445	1 1 2	2 3 4	4 5 6
70	8451	8457	8463	8470	8476	8482	8488	8494	8500	8506	1 1 2	2 3 4	4 5 6
71	8513	8519	8525	8531	8537	8543	8549	8555	8561	8567	1 1 2	2 3 4	4 5 5
72	8573	8579	8585	8591	8597	8603	8609	8615	8621	8627	1 1 2	2 3 4	4 5 5
73	8633	8639	8645	8651	8657	8663	8669	8675	8681	8686	1 1 2	2 3 4	4 5 5
74	8692	8698	8704	8710	8716	8722	8727	8733	8739	8745	1 1 2	2 3 4	4 5 5
75	8751	8756	8762	8768	8774	8779	8785	8791	8797	8802	1 1 2	2 3 3	4 5 5
76	8808	8814	8820	8825	8831	8837	8842	8848	8854	8859	1 1 2	2 3 3	4 5 5
77	8865	8871	8876	8882	8887	8893	8899	8904	8910	8915	1 1 2	2 3 3	4 4 5
78	8921	8927	8932	8938	8943	8949	8954	8960	8965	8971	1 1 2	2 3 3	4 4 5
79	8976	8982	8987	8993	8998	9004	9009	9015	9020	9025	1 1 2	2 3 3	4 4 5
80	9031	9036	9042	9047	9053	9058	9063	9069	9074	9079	1 1 2	2 3 3	4 4 5
81	9085	9090	9096	9101	9106	9112	9117	9122	9128	9133	1 1 2	2 3 3	4 4 5
82	9138	9143	9149	9154	9159	9165	9170	9175	9180	9186	1 1 2	2 3 3	4 4 5
83	9191	9196	9201	9206	9212	9217	9222	9227	9232	9238	1 1 2	2 3 3	4 4 5
84	9243	9248	9253	9258	9263	9269	9274	9279	9284	9289	1 1 2	2 3 3	4 4 5
85	9294	9299	9304	9309	9315	9320	9325	9330	9335	9340	1 1 2	2 3 3	4 4 5
86	9345	9350	9355	9360	9365	9370	9375	9380	9385	9390	1 1 2	2 3 3	4 4 5
87	9395	9400	9405	9410	9415	9420	9425	9430	9435	9440	0 1 1	2 2 3	3 4 4
88	9445	9450	9455	9460	9465	9469	9474	9479	9484	9489	0 1 1	2 2 3	3 4 4
89	9494	9499	9504	9509	9513	9518	9523	9528	9533	9538	0 1 1	2 2 3	3 4 4
90	9542	9547	9552	9557	9562	9566	9571	9576	9581	9586	0 1 1	2 2 3	3 4 4
91	9590	9595	9600	9605	9609	9614	9619	9624	9628	9633	0 1 1	2 2 3	3 4 4
92	9638	9643	9647	9652	9657	9661	9666	9671	9675	9680	0 1 1	2 2 3	3 4 4
93	9685	9689	9694	9699	9703	9708	9713	9717	9722	9727	0 1 1	2 2 3	3 4 4
94	9731	9736	9741	9745	9750	9754	9759	9763	9768	9773	0 1 1	2 2 3	3 4 4
95	9777	9782	9786	9791	9795	9800	9805	9809	9814	9818	0 1 1	2 2 3	3 4 4
96	9823	9827	9832	9836	9841	9845	9850	9854	9859	9863	0 1 1	2 2 3	3 4 4
97	9868	9872	9877	9881	9886	9890	9894	9899	9903	9908	0 1 1	2 2 3	3 4 4
98	9912	9917	9921	9926	9930	9934	9939	9943	9948	9952	0 1 1	2 2 3	3 4 4
99	9956	9961	9965	9969	9974	9978	9983	9987	9991	9996	0 1 1	2 2 3	3 3 4

Fig. 7. Table of Logarithms and Proportional Parts

are (a) grams per milliliter, (b) pounds per cubic foot, (c) pounds per gallon. It is imperative that the physical unit be written after the numerical value. Some representative densities are:

Substance	lb./ft.³
Air	0.0795
Coal, anthracite	52-60
Coke, loose	23-32
Cork	15
Ice	57.5
Iron, wrought	480
Quartz	165
Water 32°F (0°C)	62.417
Water 62°F (16.6°C)	62.355
Water 212°F (100°C)	59.7
Dry sand	90-100

For uniform, non-porous material, the density is independent of the size and shape of the sample. For non-uniform material or when voids are present or for lump material with varying particle size, the density may fluctuate widely. A single grain of white sand if expanded to a cubic foot would weigh 165 pounds.

A cubic foot of these same sand grains would weigh about 100 pounds. (165 — 100)/165 = 0.394. Here the voids occupy 39.4 per cent.

The formula D = W/V may be rearranged to give V = W/D or W = V×D.

Problem: Calculate the weight of a bar of wrought iron that is 13 ft. x 4 in. x 4 in.

$$\text{Volume (ft.}^3\text{)} = 13 \times \frac{4}{12} \times \frac{4}{12} = 1.44$$

1.44 x 480 lb./ft.³ = 691.2.

Problem: A vertical cylindrical tank is 3 ft. in diameter. The level of oil drops 2 ft. The oil has a density of 7.74 lb./gal. What weight of oil was drawn from the tank?

Volume of oil = $\pi(3/2)^2 \times 2$

$\pi(3/2)^2 \times 2 \times 7.48 \times 7.74 = 818$ lb. oil

Specific Gravity

Density and specific gravity are closely related. Specific gravity is a relative density. It is obtained by dividing the density of the object by the density of some reference substance. Water is the usual reference substance. Specific gravity is a pure number and has no physical unit. It is frequently viewed as the weight of an object divided by the weight of an equal volume of water. For its exact specification, the temperatures of object and water must be given.

Sp.G. $0.842\,{}^{20°}_{4°}$ means that the liquid was at 20° C. and the water at 4° C.

$$\text{Sp.G.} = d_o/d_w \text{ or Sp.G.} \times d_w = d_o$$

where w refers to water and o refers to the object.

Problem: The specific gravity of an oil is 0.934. What is the weight of a gallon?

0.934 x 8.34 = 7.78 lb./gal.

Problem: 93.00 per cent sulphuric acid has a Sp.G. of 1.8279. Calculate (a) the weight of one liter, (b) grams of acid per liter, (c) pounds of acid per cubic foot, (d) pounds of acid per gallon, (e) gallons of the 93 per cent acid required per hour to treat a flow of 5 million gallons per day at a rate of 2 ppm.

(a) weight of 1 liter = 1000 x Sp.G.
= 1000 x 1.8279 = 1827.9 grams

(b) grams of acid per liter = 1000 x Sp.G. x %/100
= 1000 x 1.8279 x 0.93 = 1700 grams/liter

(c) lb. acid per cubic foot = Sp.G. x 62.4 x 0.93
= 106 lb./ft.³

(d) lb. acid per gallon = Sp.G. x 8.33 x 0.93
= 14.2 lb./gal.

(e) 2 ppm. = 2 x 8.34 lb. per million gallons

5/24 = million gallons per hour
5/24 x 2 x 8.34 = lb. of 100% acid per hour

5/24 x 2 x 8.34 x 1/14.2 = gallons 93 per cent acid per hour
= 0.244 gal./hr.

Problem: A clean, empty pint jar weighs 390 grams. When completely filled with water, the weight is 863 grams. When completely filled with sludge, the weight is 877 grams. What is the specific gravity of the sludge?

863 — 390 = weight of water = 473 grams

877 — 390 = weight of sludge = 487 grams

$$\text{Sp.G.} = \frac{487}{473} = 1.03$$

Chemical Equations and Formulas

A pure, definite chemical compound is represented by a formula. Some examples of interest to water and sewage men are:

Name	Formula
Water	H_2O
Calcium carbonate	$CaCO_3$
Calcium oxide	CaO
Calcium hydroxide	$Ca(OH)_2$
Sodium carbonate	Na_2CO_3
Carbon dioxide	CO_2
Sulphuric acid	H_2SO_4
Sodium hydroxide	$NaOH$
Aluminum oxide	Al_2O_3
Aluminum sulphate	$Al_2(SO_4)_3$
Copper sulphate	$CuSO_4.5H_2O$

The symbols in the formulas represent definite, fixed weights of the elements. These weights are found in a Table of Atomic Weights in most reference books. The per cent composition of a pure compound may be calculated if the formula is known.

Problem: Calculate the per cent by weight of aluminum in pure Al_2O_3.

From a Table of Atomic Weights, Al = 26.97 and O = 16.00

therefore $Al_2O_3 = 2 \times 26.97 + 3 \times 16.00 = 101.9$

This 101.9 = the molecular weight of Al_2O_3

This 101.9 parts of Al_2O_3 contains 2 x 26.97 parts of aluminum.

$$\frac{53.94}{101.9} \times 100 = 52.9\%$$

Frequently, commercial chemicals are not pure substances. Calculations are first made as if they were pure and then a correction is applied. If some situation required 40 lb. of pure CaO and the material available contains 85 per cent CaO, then 100/85 x 40 = lb. actual material to be used.

Chemical Equations

A chemical change, if understood, is represented by a chemical equation.

$$\underset{}{\overset{74.10}{Ca(OH)_2}} + \overset{98.08}{H_2SO_4} = \overset{136.1}{CaSO_4} + \overset{36.04}{2H_2O}$$

The chemical equation shows what substances disappear (on left) and what substances appear (on right). Since the symbols have definite weight meanings, the equation represents weight ratios in which chemical substances react or are produced. The numbers above the formulas are the sums of the atomic weights for the symbols shown. The equation states that 74.10 units of weight of calcium hydroxide will neutralize 98.08 units of weight of sulphuric acid.

Problem: What weight of hydrated lime, $Ca(OH)_2$, will neutralize 40 lb. of sulphuric acid?

74.10/98.08 x 40 = 30.2 lb.

Frequently chemical changes in water and sewage are only partially understood. Chemical equations may be written to represent some aspects of the reactions. Calculations are made in spite of inadequate knowledge and serve as a starting point for the practical adjustment of some process. For example, the net result of adding alum to water may be formally represented by

$$\overset{342.1}{Al_2(SO_4)_3} + \overset{300.3}{3CaCO_3} + 3H_2O = 2Al(OH)_3 + 3CaSO_4 + 3CO_2$$

This equation shows that 342.1 grains of pure anhydrous aluminum sulphate removes 300.3 grains of calcium carbonate or 300.3/342.1 = grains $CaCO_3$ per grain $Al_2(SO_4)_3$

Filter alum as used is approximately 57 per cent $Al_2(SO_4)_3$

300.3/342.1 x 0.57 = grains $CaCO_3$ per grain actual alum.

1 grain per gallon = 17.1 ppm.

therefore $\frac{17.1 \times 300.3 \times 0.57}{342.1}$ = 8.56 ppm. alkalinity removed per 1 grain per gallon of alum.

The actual reduction of alkalinity caused by the addition of alum is not far from this figure.

Solutions—Part A

Definitions and General Relations

A solution is a homogeneous system of variable composition. One of the two components is arbitrarily called the solute and the other the solvent. When alum is dissolved in water, the alum is the solute and the water the solvent. In practice the strength or composition of the solution must be stated. This is done by means of various general ratios.

(a) $\frac{\text{wt. of solute} \times 100}{\text{wt. of solution}}$

(b) $\frac{\text{wt. of solute}}{\text{volume of solution}}$

(c) $\frac{\text{wt. of solute}}{\text{wt. of solvent}}$

The general ratios take on different numerical values according to the units of weight and volume. Some examples are wt. %, lb./ft.^{3}, grams/liter, mols/liter, equivalents/liter, grains/gal., and ppm.

In the metric system with water as the solvent and for dilute solutions, the three general ratios above are numerically close together. In order to convert (b) into (a) or (c) and vice versa, the specific gravity must be known.

Problem: A solution of ammonia in water contains 28 per cent ammonia. Its specific gravity is 0.8980.

(a) Calculate lb. of ammonia per cubic foot of solution.

Water = 62.4 lb./ft.^{3}
1 ft.^{3} of solution weighs 62.4 x 0.8980 lb.
28 per cent means 28/100 lb. of ammonia per lb. of solution 62.4 x 0.8980 x 28/100 = 15.7 lb. ammonia per cubic foot.

(b) Calculate the grams of ammonia per liter of solution.

weight of 1 liter of solution = 1000 x 0.8980
1000 x 0.8980 x 28/100 = 251.4 grams ammonia per liter.

(c) Calculate the mols of ammonia per liter.

The formula for ammonia is NH_3. The atomic weights are: N = 14.008 and H = 1.008. The formula weight is 17.03. The grams per liter from (b) is 251.4.

251.4/17.03 = 14.76 mols/liter. This is the molarity of the solution. Since the formula weight in this case is also the equivalent weight, for ammonia, the 14.76 is also the normality.

(d) Calculate the grains per gallon.

Section (a) gave 15.7 lb./ft.^{3}
1 lb. = 7000 grains
1 ft.^{3} = 7.48 gallons
15.7/7.48 lb. ammonia per gallon = 2.10
2.10 x 7000 = 14,700 grains per gallon

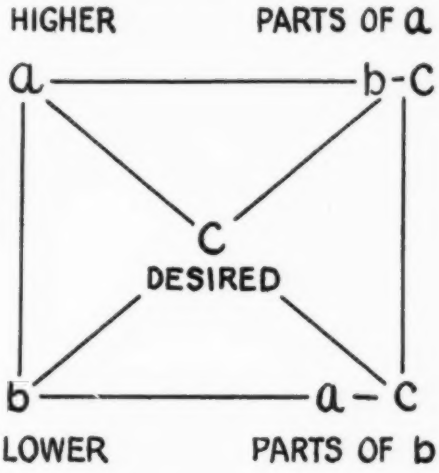
Solutions—Part B

The Dilution Rule

In handling or preparing solutions, a frequent problem is to determine the ratio in which water and a stock solution or two stock solutions are to be mixed to prepare a solution of desired composition. The general law for mixtures is

$$Q_1R_1 + Q_2R_2 = Q_mR_m$$
$$Q_1 + Q_2 = Q_m$$

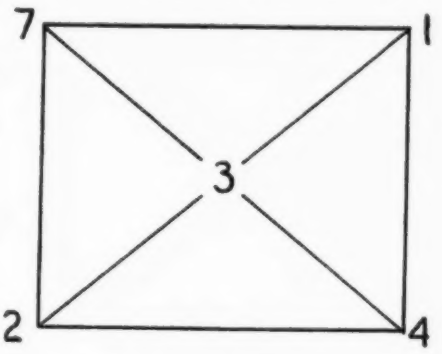
The Q's are quantities in either weight or volume. The R's are concentrations in any units. The subscripts 1 and 2 refer to the solutions to be mixed and the m to the mixture. The simplest way to make the calculation is by the "Rectangle Method," sometimes called the "Dilution Rule."



where a = the higher concentration
b = the lower concentration
c = the desired concentration

The use of this Rectangle Method is shown in the following problems.

Problem: What weights of a 2 per cent and a 7 per cent solution must be mixed to make 500 lb. of a 3 per cent solution?



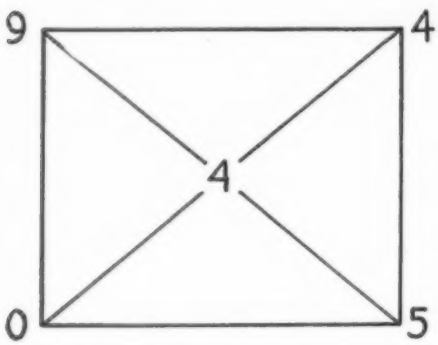
1 and 4 are wt. ratios
1 + 4 = 5

Observe where the percentages for the two stock solutions and the desired per cent are placed. Subtract along the diagonal as shown. One part by weight of the 7 per cent and 4 parts by weight of the 2 per cent solutions are to be mixed. This will give 5 parts by weight of a 3 per cent solution.

1/5 × 500 = wt. of 7 per cent solution needed
4/5 × 500 = wt. of 2 per cent solution needed

This method is correct for all weight-per cent compositions.

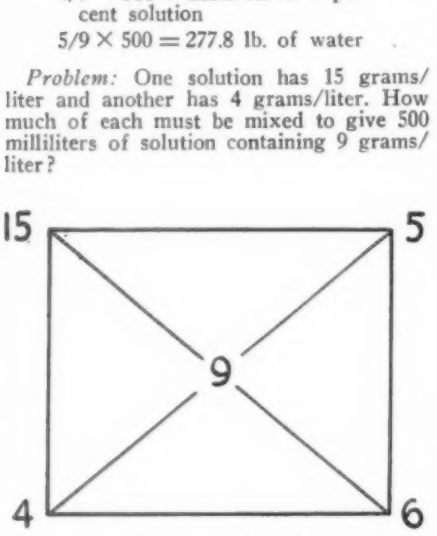
Problem: What weight of water and a 9 per cent solution must be mixed to make 500 lb. of 4 per cent solution?



4 and 5 are wt. ratios
$4 + 5 = 9$

$4/9 \times 500 = 222.2$ lb. of 9 per cent solution
$5/9 \times 500 = 277.8$ lb. of water

Problem: One solution has 15 grams/liter and another has 4 grams/liter. How much of each must be mixed to give 500 milliliters of solution containing 9 grams/liter?



5 and 6 are vol. ratios
$5 + 6 = 11$

$5/11 \times 500 = 227.3$ ml. of the 15 grams/liter solution
$6/11 \times 500 = 272.7$ ml. of the 4 grams/liter solution

The method is accurate for dilute solutions. It is strictly correct as long as $V_1 + V_2 = V_m$. This is sometimes not correct for concentrated solutions.

Solutions—Part C

Chemical Formulas and Equations

A pure, definite chemical substance is represented by a formula. $CaCO_3$ represents pure calcium carbonate. The chemical symbols in practical work represent definite weights of the elements. These weights are found in a table of Atomic Weights. Ca means 40.08 parts of calcium. C stands for 12.00 parts of carbon, and O represents 16.00 parts of oxygen. Since these weights are relative, any unit of weight may be used as long as the same unit is used throughout the calculation. If the symbol weights in $CaCO_3$ are added, we obtain the formula weight or molecular weight. This is 100.0 for $CaCO_3$. This is also called one "chemical unit" or one mol.

Consider the formula for pure hydrogen chloride, HCl. This represents one mol with a weight of $1.008 + 35.457 = 36.465$. In this case, the weight of one mol is also called "one equivalent weight." In any chemical reaction, each equivalent weight of a reactant consumes exactly one equivalent weight of any other reactant. When hydrochloric acid reacts with calcium carbonate, the following equation gives the weight ratios.

$$CaCO_3 + 2HCl = CaCl_2 + CO_2 + H_2O$$

Since 1 HCl has just been defined as 1 equivalent weight, 2 HCl must be 2 equivalent weights. Therefore $CaCO_3$ must be two equivalent weights or one equivalent weight of $CaCO_3$ must be ½ $CaCO_3$.

Since $NaOH + HCl = NaCl + H_2O$, it follows that 1 NaOH must be 1 equivalent weight.

Also $2NaOH + H_2SO_4 = Na_2SO_4 + 2H_2O$

and therefore 1 H_2SO_4 must be two equivalent weights. When the reaction involves a change in valence of the principle elements, the equivalent weight is found by dividing the molecular weight by the change in valence.

A table of some common chemicals used in water and sewage works laboratories follows with the equivalent weight formula:

Substance	Equivalent Weight
HCl	HCl
H_2SO_4	$\frac{1}{2}H_2SO_4$
$CaCO_3$	$\frac{1}{2}CaCO_3$
CaO	$\frac{1}{2}CaO$
$Ca(OH)_2$	$\frac{1}{2}Ca(OH)_2$
NaOH	NaOH
Na_2CO_3	$\frac{1}{2}Na_2CO_3$
$NaHCO_3$	$NaHCO_3$
$AgNO_3$	$AgNO_3$
$KMnO_4$	$1/5KMnO_4$
$H_2C_2O_4$	$\frac{1}{2}H_2C_2O_4$
$Na_2S_2O_3 \cdot 5H_2O$	$Na_2S_2O_3 \cdot 5H_2O$

A normal solution contains one equivalent weight in each liter of the solution. Two reacting solutions of the same normality react volume for volume.

A useful concept is the milliequivalent weight. This is 1/1000 of the equivalent weight. The normality of a solution is most usefully defined as the number of milliequivalents per milliliter.

Problem: How many ml. of 93 per cent H_2SO_4, Sp.G. 1.84, are required to make 1 liter of 0.0200 N H_2SO_4?

The equivalent wt. of sulphuric acid from the above table is $\frac{1}{2}H_2SO_4$

The molecular weight of $H_2SO_4 = 98.02$

$98.02 \times \frac{1}{2} \times 0.0200$ = wt. of pure acid in each liter of 0.0200 normal solution

$1.00 \times 1.84 \times 93/100$ = wt. of pure acid in 1 ml. of stock solution

$$\frac{98.02 \times \frac{1}{2} \times 0.0200}{1.84 \times 93/100} = \text{ml. of stock to be diluted to 1 liter}$$

After solutions are prepared, it is usually necessary to standardize them by some accurate method of analysis.

Problem: 0.2473 grams of Na_2CO_3 was used to standardize an unknown solution of H_2SO_4. 20.08 ml. of the acid solution was required to reach the methyl orange point. What is the normality of the acid?

The milliequivalent wt. of sodium carbonate is $Na_2CO_3/2000$

$Na_2CO_3/2000 = 0.0530$ grams

0.2473/0.0530 = milliequivalent of Na_2CO_3 taken = 4.666

$4.666/20.08 = 0.2326$ = normality

Problem: What volume of 0.2326 N acid must be diluted to exactly 1 liter in order that the resulting solution be exactly 0.0200 N?

When a solution is diluted with water
$V_1N_1 = V_2N_2$
$V_1 \times 0.2326 = 1000 \times 0.0200$
$V_1 = 85.98$ ml.

Problem: 100 ml. of water required 11.20 ml. of 0.0200 N acid to reach the methyl orange end-point. Calculate the alkalinity of the water in ppm. of $CaCO_3$.

mew = grams in 1 milliequivalent weight
N = normality

$$\frac{\text{ml.} \times N \times \text{mew} \times 1000}{\text{liters of sample}} = \text{ppm. } CaCO_3$$

$$\frac{11.2 \times 0.0200 \times 2000 \times 1000}{0.1} = 112.1 \text{ ppm.}$$

The titer of a solution is the grams of the substance sought represented by 1 ml. of the reagent. It is calculated by multiplying the normality of the reagent used by the milliequivalent weight of the substance sought.

Solutions—Part D

Factor Solutions

When the same titration analysis is to be repeated many times, it is a convenience to adjust the strength of the reagent or to take such a sample that the calculations can be made from the burette reading by simply multiplying by 1, 10, 0.1 etc. Reagents so adjusted are called "Factor Solutions." For a fixed sample of 100 ml, this is done as follows: the general formula for ppm. is

$$\frac{\text{ml.} \times N \times \text{mew} \times 1000}{0.100} = \text{ppm.}$$

Let us specify that each ml. of acid in an alkalinity determination on 100 ml. sample shall represent 10 ppm. of $CaCO_3$

$$\frac{1 \times N \times 2000 \times 1000}{0.100} = 10 \text{ ppm. } CaCO_3$$

Here N is calculated to be 0.01998 normal.

Problem: What must be the normality of a thiosulphate solution if each ml. of the solution used in the titration on 100 ml. of sample shall represent 2 ppm. of dissolved oxygen?

$$\frac{\text{ml.} \times N \times \text{mew} \times 1000}{0.1}$$

0.008 = mew of oxygen

$$\frac{1 \times N \times 0.008 \times 1000}{0.1} = 2$$

$N = 0.025$

Elementary Hydraulics

Head

Pressure at depth h below the surface of a liquid is given by

$P = dh$ where
P = pressure in lb./ft.²
d = density in lb./ft.³
h = depth in ft.

Then lb./ft.² = lb./ft.³ x ft.
d = 62.34 lb./ft.³ for water at 60° F.

P in lb./in.² = 1/144 x lb./ft.²
P in lb./in.² = 0.433 x ft. depth

Feet depth = P in lb./in.² x 2.31.

The feet of water necessary to produce a specified static pressure is called the "head." The term head has a more gen-

eral and useful meaning. Strictly interpreted, it means foot-pounds of energy per pound of fluid.

Energy is that which can be converted into heat. There are only two fundamental forms of energy (a) energy due to motion, (b) energy due to position when force is acting. Energy of motion is kinetic and energy of position is potential.

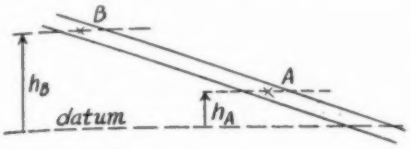


Fig. 8. Diagram Showing Head Relations at Different Points

Consider Fig. 8, in which water is flowing in a pipe. A and B are two stations on the line. The datum is a horizontal reference plane. Each pound of water at B is elevated h_B feet above the datum plane and is acted upon by a pound of force due to gravity. Each pound of water elevated h feet possesses h ft.-lb. of potential energy and is said to have a potential "head" of h feet. If the water at B is moving, it has kinetic energy in ft.-lb. equal to $V^2/2g$ for each lb. V = velocity in ft./sec. and g (gravity) is 32.2 ft./sec./sec. This kinetic energy in ft.-lb. per lb. of water is the "velocity head."

In addition, the water at B exerts a static pressure equal to P/d. This latter is the "pressure head."

Two other heads are usually involved when water flows. When any liquid flows, there is friction. Friction in a moving liquid means that energy is being converted into heat. If the potential, velocity and pressure heads are added for a liquid at a given point, we have the total ft.-lb. of energy per lb. of liquid at that point. As this pound of liquid moves, it steadily loses energy through friction. This is what is meant by "loss of head."

Every length of pipe, channel, elbow, valve, baffle, or anything that increases turbulence or any sudden change of cross-section or orifice, is responsible for loss of head. This loss of head due to friction is permanent.

A pump is used primarily to increase available head and is a device for adding ft.-lb. energy to each lb. of water passing through. We can properly speak of "friction head" and "pump head." Friction heads for pipe and fittings are readily obtained from manufacturers' bulletins and data books. (See *Water Works & Sewerage*, Vol. 91, p. R-28, June, 1944, for a complete discussion of Loss of Head.)

The various heads along a hydraulic system are not independent. They are related by Bernouilli's Theorem. Consider Fig. 8. Points B and A are on a straight pipe of uniform cross-section. The horizontal datum plane may be taken anywhere. Water is flowing from B to A.

$$h_B + \frac{V^2_B}{2g} + \frac{p_B}{d} - F = h_A + \frac{V^2_A}{2g} + \frac{p_A}{d} \qquad \text{Eq. (1)}$$

This equation simply states that the sum of the potential, velocity, and pressure heads at B minus the friction head is equal to the sum of the three heads at A. Since the cross-sections at A and B are equal, $V_B = V_A$ and the equation becomes

$$h_B + \frac{p_B}{d} - F = h_A + \frac{p_A}{d} \qquad \text{Eq. (2)}$$

If A and B were at the same elevation, h_A would equal h_B and the equation would become

$$\frac{p_B}{d} - F = \frac{p_A}{d} \text{ or}$$

$$\frac{p_B}{d} - \frac{p_A}{d} = F \text{ feet} \qquad \text{Eq. (3)}$$

Equation (3) tells us that the pressure drop on a horizontal line of uniform cross-section is due entirely to friction.

Venturi Meter

In the Venturi meter, the throat at B has a smaller cross-section than at A. Be-

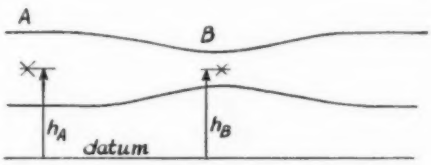


Fig. 9. Venturi Meter

cause the same quantity of water flows through both sections, we have

$$(\text{velocity} \times \text{area})_A = (\text{velocity} \times \text{area})_B$$

The velocity at B is greater than at A. From Bernouilli's theorem

$$h_A + \frac{V^2_A}{2g} + \frac{p_A}{d} - F = h_B + \frac{V^2_B}{2g} + \frac{p_B}{d}$$

$h_A = h_B$
F = small

$$\frac{p_A}{d} - \frac{p_B}{d} = \frac{V^2_B}{2g} - \frac{V^2_A}{2g} \text{ or}$$

omitting friction

$$\sqrt{V^2_B - V^2_A} = \sqrt{2g\left(\frac{p_A}{d} - \frac{p_B}{d}\right)}$$

including friction

$$\sqrt{V^2_B - V^2_A = 0.98} \sqrt{2g\left(\frac{p_A}{d} - \frac{p_B}{d}\right)}$$

$$\frac{V_A}{V_B} = \frac{\text{area B}}{\text{area A}}$$

The last two equations show that the pressure drop between A and B and the two areas enable the velocity of flow to be calculated.

Problem: A 2 in. pipe on a horizontal run of 100 ft. has a flow of 80 gal./min. The pressure drop is 10.3 lb./in.^{2} Calculate the head lost in friction and the horsepower consumed in this length of pipe.

lb./in.^{2} x 144 = lb./ft.^{2}
lb./ft.^{2} of pressure drop ÷ density = ft. head lost

10.3 x 144 x 1/62.4 = 23.8 ft. head lost

This lost head means that for every pound of water passing through this length of pipe, 23.8 ft. = lb. of energy go into heat.

80 gal./min. = 80 x 8.33 lb./min.
80 x 8.33 x 23.8 = ft.-lb./min.
33000 ft.-lb./min. = 1 horsepower

$$\frac{80 \times 8.33 \times 23.8}{33000} = 0.481 \text{ horsepower}$$

Problem: A 2 in. pipe delivers 80 gal./min. Calculate the linear velocity of flow in ft./sec.

80 gal./min. = 80 x 1/7.48 x 1/60 ft.^{3}/sec.
cross-section = 3.1416 x (1/12)^2 = ft.^{2}
ft.^{3}/sec. divided by ft.^{2} = ft./sec.
80 x 1/7.48 x 1/60 x 144 x 1/3.1416 = 8.17 ft./sec.

Problem: The influent valve on a filter is closed. The water surface requires 4.5 minutes to drop 1 foot. Calculate the rate of filtration in gal./min. per ft.^{2} of sand surface.

Rate of filtration is 1 cubic foot in 4.5 minutes per ft.^{2}
7.48/4.5 = 1.67 gal./min. per ft.^{2}

If the sand area is 30 × 15 ft., what is the 24 hour rate in millions of gallons?

$$\frac{1.67 \times 60 \times 24 \times 30 \times 15}{1{,}000{,}000} = 1.077 \text{ mgd.}$$

What is the rate in millions of gallons per acre per day?

$$\frac{30 \times 15}{43560} = \text{acres} = 0.0103$$

$$\frac{1.077}{0.0103} = 104.3 \text{ mgad.}$$

Averages and Means

Arithmetic Mean

An average is a single number used to represent a group of measurements or data. Averages are of different kinds. The commonly used average is the *arithmetic mean.* It is what is usually meant by the "mean" or "average." If X, etc., are the values of the individual measurements and N is their number,

$$\text{arithmetic mean} = \overline{X} = \frac{X_1 + X_2 + X_3 + \text{etc.}}{N}$$

If X_1 appears f_1 times, etc.,

$$\overline{X} = \frac{X_1 f_1 + X_2 f_2 + X_3 f_3 + \text{etc.}}{N}$$

where N = sum of all f's

Weighted Averages

This last equation shows that each X must be weighted according to the number of times that it occurs. In the following table the rate of pumping is constant beginning on the hour and continuing until the next recorded hour.

Hour	mil. gal./hr.
12 midnight	2.0
1 A.M.	2.5
3 A.M.	2.7
6 A.M.	3.0
7 A.M.	7.0
12 noon	6.0
2 P.M.	4.0
5 P.M.	3.0
9 P.M.	2.0

The average desired is the mil. gal. per hour over the 24 hour period. If we merely added the values in the table and divided by 9, we would get 3.58. This would not be correct because the rates given in the table did not hold for equal times. Each rate must be weighted according to the hours run at that rate.

2.0 x 1 = 2.0
2.5 x 2 = 5.0
2.7 x 3 = 8.1
3 x 1 = 3.0
5 x 7 = 35.0
6 x 2 = 12.0
4 x 3 = 12.0
3 x 4 = 12.0
2 x 3 = 6.0
Sum = 95.1

$$\text{Weighted average} = \frac{95.1}{24} = 3.96 \text{ gal. per hr. for the correct average.}$$

The Median

Another type of average is the *median.* This is determined by arranging the values in descending order and selecting the value

half-way down. Another method of obtaining the mean is to cross out the largest and smallest values, continuing until only one is left. If two are left, the average of these two is taken. Half the values are larger than the median and half are smaller. It has the advantage of not being affected by an abnormally large or small value of a single quantity.

The Mode

Another representative value is the *mode*. The mode is that value which in any series of observations occurs more often than any other. Its chief use in sanitation studies is in connection with the M.P.N. (Most Probable Number) values for Coliform density. In general operation practice it is not customary to use the mode. Arithmetic averages are commonly used.

In the determination of the arithmetic mean M.P.N. for a month, it must be remembered that the M.P.N. for each day or sample must be determined and the sum of the M.P.N.'s divided by the total number of days or samples. It is not permissible to average the tube readings for the month and determine the M.P.N. therefrom.

Quantity Units

In sanitation studies, the characteristics of a water or sewage are usually expressed in concentration units. A common concentration unit is parts per million (ppm.). For some studies, the total pollutional load or some figure that is proportional to this quantity is desirable. For this purpose, *quantity units* are used. A quantity unit involves both concentration and total flow. When applied to the biochemical oxygen demand, a convenient unit would be the lb. of B.O.D. per day in the total flow.

ppm. x 8.34 x mgd. = lb. B.O.D. per day
or ppm. x cfs. x 5.4 = lb. B.O.D. per day

An arbitrary unit ppm. × cfs. is used by some workers.

A quantity unit used in the bacteriological study of polluted streams is the total bacteria passing through a square-foot section in 24 hours when the velocity of flow is 1 ft. per sec. and the organisms are 1,000 per ml.

$$1000 \times 28{,}350 \times 86{,}400 = 2.45 \times 10^{12}$$
1 Q.U. here is 2.45×10^{12}

Another simpler arbitrary unit is

$$\frac{\text{organisms per ml.}}{1000} \times \text{cfs.}$$

For comparison purposes, arbitrary units may be used as long as they involve both concentration and flow.

In sewage works, pounds of suspended solids removed per day, pounds of dry sludge solids per day, and pounds of B.O.D. removed per day are all quantity units. From day to day, per cent removals may be calculated from concentrations (ppm. or bacteria per ml.), but the average per cent removal may be calculated only from the *weighted average* concentration or from quantity units. It cannot be determined from the arithmetic mean of the per cent removal. Unless flows are constant from day to day or approximately so the average concentration (ppm.) is correct only when determined by weighting i.e., from quantity units.

Drying and Digestion of Sludge

Problem: Sludge run onto a sand bed contained 92 per cent water and after drying contained 70 per cent water. What per cent of the water originally present was removed?

The wet sludge contains 92 lb. of water to 8 lbs. of dry solids. The dried sludge contains 70 lb. of water to 30 lb. of dry solids.

92/8 — 70/30 = lb. of water removed per lb. of dry solids.

11.50 — 2.33 = 9.17.

9.17/11.50 x 100 = 79.7% removal.

Problem: Completely dried raw sludge contained 75 per cent volatile matter. The completely dried digested sludge contained 60 per cent volatile matter. What per cent of the volatile matter was removed; i.e., what was the per cent digestion?

100 lb. of the raw dry sludge contains 75 lb. volatile matter and 25 lb. of ash.

75/25 = lb. volatile per lb. of ash for the raw sludge.

60/40 = lb. volatile per lb. non-volatile for the digested sludge.

75/25 — 60/40 = 3.0 — 1.5 = 1.5.

$\frac{1.5}{3} \times 100 = 50\%$ digestion of vol. mat.

TABLE FOR DETERMINING MOST PROBABLE NUMBERS OF COLIFORM BACTERIA

Most Probable Numbers per 100 ml. of Sample. Tests by Fermentation Tube Method Using Three 10 ml. Tubes, Three 1 ml. Tubes and Three 0.1 ml. Tubes.

No. ml. 10.	Positive ml. 1.0	Tubes ml. 0.1	MPN	No. ml. 10.	Positive ml. 1.0	Tubes ml. 0.1	MPN	No. ml. 10.	Positive ml. 1.0	Tubes ml. 0.1	MPN	No. ml. 10.	Positive ml. 1.0	Tubes ml. 0.1	MPN
0	0	0	-----	1	0	0	3.6	2	0	0	9.1	3	0	0	23
0	0	1	3.0	1	0	1	7.2	2	0	1	14.0	3	0	1	39
0	0	2	6.0	1	0	2	11.0	2	0	2	20.0	3	0	2	64
0	0	3	9.0	1	0	3	15.0	2	0	3	26.0	3	0	3	95
0	1	0	3.0	1	1	0	7.3	2	1	0	15.0	3	1	0	43
0	1	1	6.1	1	1	1	11.0	2	1	1	20.0	3	1	1	75
0	1	2	9.2	1	1	2	15.0	2	1	2	27.0	3	1	2	120
0	1	3	12.0	1	1	3	19.0	2	1	3	34.0	3	1	3	160
0	2	0	6.2	1	2	0	11.0	2	2	0	21.0	3	2	0	93
0	2	1	9.3	1	2	1	15.0	2	2	1	28.0	3	2	1	150
0	2	2	12.0	1	2	2	20.0	2	2	2	35.0	3	2	2	210
0	2	3	16.0	1	2	3	24.0	2	2	3	42.0	3	2	3	290
0	3	0	9.4	1	3	0	16.0	2	3	0	29.0	3	3	0	240
0	3	1	13.0	1	3	1	20.0	2	3	1	36.0	3	3	1	460
0	3	2	16.0	1	3	2	24.0	2	3	2	44.0	3	3	2	1100
0	3	3	19.0	1	3	3	29.0	2	3	3	53.0	3	3	3	-----

VALVE AND PRIMER CORPORATION

30 WEST WASHINGTON ST., CHICAGO 2, ILL.

Apco Automatic Pump Primers *Apco Automatic Air Relief Valves*

V-Apco Automatic Vacuum Primers

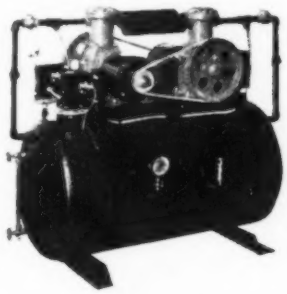
V-APCO Automatic Vacuum Primers and APCO Automatic Pump Primers

Advantages

V-APCO Automatic Vacuum Primers for centrifugal pumps offer these advantages: 1) Keep pumps always primed. 2) End loss of time due to lack of prime. 3) Operate at any vacuum under which the centrifugal pump will operate. 4) Uses no foot valve. 5) Flexible capacity; will serve additional pump installations satisfactorily. 6) Does not affect pump efficiency. 7) Improves operation of pump. 8) Simple design insures long life, freedom from trouble, highest efficiency.

How the Primer Operates

Drawing below shows the V-APCO Primer. A priming valve of the 200 series is placed on the vent of the centrifugal pump. While the vacuum pump is producing a vacuum, the valve passes all air that



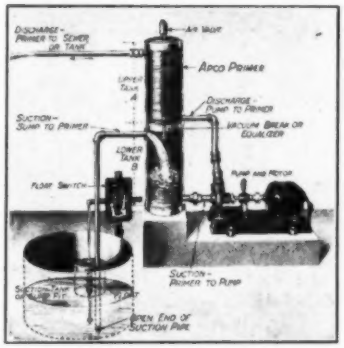
may be in the pump or suction line. When the air is withdrawn, it is followed by water which raises the valve float. That closes the outlet and prevents flooding of the vacuum pumps.

The Air Release Valve is mounted on the pump and connected to the vacuum storage tank that stores enough vacuum to act as a cushion on the vacuum pump. It provides a way to withdraw small amounts of air from the pump and suction lines from time to time. This makes frequent starting and stopping of the vacuum pump unnecessary.

A vacuum breaker on the vacuum tank unloads the vacuum pump in the unlikely event that is required. A vacuum switch with vacuum release connections is mounted on the vacuum tank and starts the vacuum pump when the vacuum drops in the storage tank and stops the pump when full vacuum has been reached. The vacuum release connection unloads the vacuum pump when it is not operating and places it in condition to start at no load.

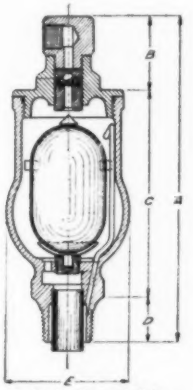
The check valve between the switch and the pump is a special noiseless one for the purpose, holding the vacuum in the storage tank when the pump is not operating.

THE APCO PRIMER

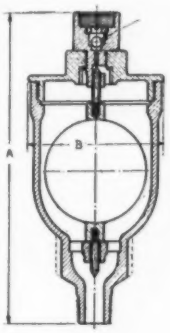


How the Apco Primer Works

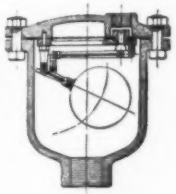
This cut shows primer in service with the pump running. When the pump starts, it draws liquid from the lower chamber of the primer and discharges it through the upper chamber. The withdrawal of the liquid from the lower chamber creates a partial vacuum in this chamber which causes the liquid in the sump or well to rise in the suction pipe and flow through the primer to the pump. When the pump stops, the liquid in the upper chamber runs back through the pump into the lower chamber by gravity, thus refilling the lower chamber with liquid to prime the pump when it starts again.



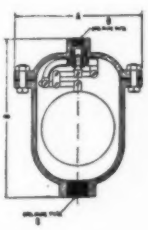
Sectional View of Valves Nos. 50-60 for Pressures Up to 60 Lbs.



Sectional View of Valve No. 50 H.D. for Pressures up to 150 Lbs.



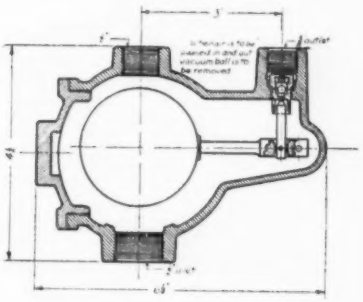
Sectional view of Valve No. 200. The valve has large venting capacities and positive opening when under vacuum.



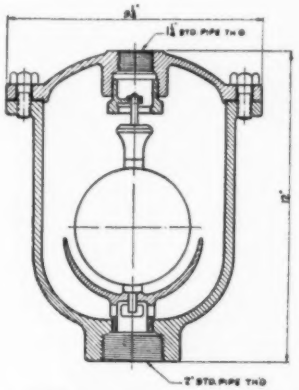
Sectional view of Valve No. 200-A.

APCO-Air Release Valves

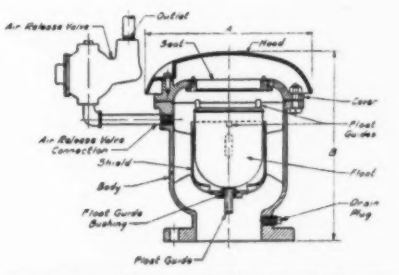
APCO Air Release Valves are made in 17 different sizes and capacities varying from the smallest with ⅜" pipe connection and small venting capacity to our large 2" size with an opening of 1" diameter and vacuum and air release valve from 2" to 6". We illustrate below a very few of these types. Your venting problems can be handled by some one of our valves in the proper manner. Prices are reasonable.



Sectional View of Valve No. 55.



Sectional View of Valve No. 140 for Deep Well Pumps.



Sectional View of No. 150 Series Air and Vacuum Valves.

THE Atlas Mineral PRODUCTS COMPANY OF PENNA.

MERTZTOWN 12, PENNA.

*ATLANTA 3, Ga., 452 Spring St., N. W. *DETROIT 2, Mich., 2970 W. Grand Blvd. PITTSBURGH 27, Pa., 4921 Plymouth Rd.
*CHICAGO 1, Ill., 333 No. Michigan Ave. NEW YORK 16, N. Y., 280 Madison Ave. ST. LOUIS 8, Mo., 4485 Olive St.

THE ATLAS MINERAL PRODUCTS CO. OF TEXAS, INC., Box 252, Houston 1, Texas

DALLAS 5, Tex., 3921 Purdue St. *LOS ANGELES 12, Calif., 172 S. Central Ave. SAN FRANCISCO 7, Calif., 115 Townsend St.
*DENVER 2, Colo., 1921 Blake St. NEW ORLEANS 12, La., 208 Vincent Bldg. SALT LAKE CITY 11, Utah, 925 S. 6th West St.
*HONOLULU 2, Hawaii, U.S.A., OMAHA, Neb., 423 South 38th Avenue *SEATTLE 4, Wash., 1252 First Avenue, S.
*KANSAS CITY, Mo., 422 B.M.A. Bldg. OKLAHOMA CITY 2, Okla., 708 Braniff Bldg. TUPELO, Miss., 517 South Spring St.

IN CANADA: Atlas Products are manufactured by H. L. BLACHFORD, Limited, 977 Aqueduct Street, Montreal, P. Q., 86 Bloor Street W., Toronto, Ont. *Stock carried at these points

Tegul-MINERALEAD JOINTING COMPOUND

FOR BELL & SPIGOT WATER MAIN

Tegul-MINERALEAD is a radically different jointing compound with advantages not matched in total in any other product for its purpose. Its sulphur base includes, as a plasticising agent, a special olefine polysulphide which contributes these advantageous properties: GREATER ADHESION TO METALLIC AND CERAMIC SURFACES • HIGHER RESISTANCE TO VIBRATION and to MECHANICAL SHOCK AND TEMPERATURE CHANGES • GREATER PLASTICITY and LOWER COEFFICIENT OF EXPANSION.

Using *Tegul*-MINERALEAD you may expect:

SPEED — *Tegul*-MINERALEAD melts, pours and sets quickly and uniformly (a factor in reducing labor costs). Joints show less than usual initial leakage and backfilling may start at once, thus hastening removal of traffic hazards which arise from open trenches.

CONVENIENCE — Molded in 10 lb. ingots, *Tegul*-MINERALEAD is easier to handle, ship and store. The ingot is impervious to moisture so may be stored outdoors in any weather. Since there can be no change of composition in an ingot, *Tegul*-MINERALEAD reaches you as correctly mixed as when it left our factory.

ECONOMY — *Tegul*-MINERALEAD weighs only 1/5th as much as lead (and goes 3 times as far) saving in shipping costs. 5 ingots to carton makes a handy package. It needs no caulking or deep bell holes and requires no skilled labor. The sum of *Tegul*-MINERALEAD'S advantages make it a *money-saver*, too.

RESISTANCE TO SHOCK — Higher resistance to mechanical and thermal shock promises permanently tight joints. Lines laid with *Tegul*-MINERALEAD in every kind of terrain and meeting every kind of punishment, have served for years without need of attention.

AVAILABILITY — *Tegul*-MINERALEAD is stocked at conveniently located points and is usually available on short notice. Using *Tegul*-MINERALEAD helps conserve lead, a scarce material.

HOW *Tegul*-MINERALEAD IS USED

MELTING — *Tegul*-MINERALEAD melts at 248°F., requiring only a slow fire. Care should be taken to avoid direct contact with flame and overheating, which tends to cause the compound to thicken. Best results are had by using furnace designed by the manufacturers of *Tegul*-MINERALEAD.

YARNING — The pipe at Bell and Spigot must be clean and free from accumulations of oil, tar and grease. If jute is employed it should be the dry, braided type.

POURING — See that joints are clean. Use mud on asbestos runner to prevent adhesion to the *Tegul*-MINERALEAD and to seal any leaks. Pour the melted compound slowly through a high pouring gate, using ladles or pouring pot of sufficient capacity to fill the joint at one operation. Add whole or half ingots to replace *Tegul*-MINERALEAD used. It is not necessary to wait for complete melting before pouring more joints. Cut off pouring gate when compound has solidified. Remove runner and joint is complete.

G-K The *Original* Bituminous SEWER JOINT COMPOUND

For Vitrified Tile and Concrete Sewer Pipe

Positive, Pliable and Permanent as vitrified tile itself. Never becomes brittle. Always retains flexibility. Needs no skilled labor. Can be used in winter temperatures. By preventing infiltration of ground water, G-K makes subdrains unnecessary. Completely fills annular space between bell and spigot. Standard package 170 lbs.

The HYDE-RO Ring

AN IMPORTANT AID IN BELL & SPIGOT JOINTING

***SPEEDS STERILIZATION*—First hypochlorite flushing provides sterile joint.**

Cross section

PROMOTES ECONOMY — FACILITATES REPAIRS EASILY APPLIED

ADVANTAGES IN DETAIL . . .

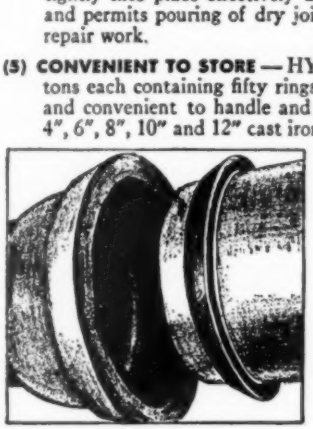
(1) PREVENTS CONTAMINATION — Rubber cannot harbor bacteria nor support their growth internally where water main sterilizing agents cannot reach them. The use of HYDE-RO Rings will prevent continued recontamination that has been traced in many cases to the joint packing.

(2) ECONOMICAL — The cost of HYDE-RO Rings is insignificant compared with the cost of repeated treatments with hypochlorite or the cost of sterilizing and keeping sterile (if it can be done), other types of packing.

(3) CONVENIENT — The installing of HYDE-RO Rings is a simple, easily understood operation. There is no measuring to be done and no cutting. The wrong size cannot be used. Since rings are solid, there are no laps to be inspected to see that they do not occupy too much of the joint space. The same ring fits pipe with bead on spigot or straight end pipe. The narrow iron blade used in caulking the ring automatically slips into the groove in the back.

(4) HOLDS BACK WATER — When HYDE-RO Rings are used, the pouring of wet joints is unnecessary. The rubber ring caulked tightly into place effectively dams off water from leaking valves and permits pouring of dry joints. This is especially important in repair work.

(5) CONVENIENT TO STORE — HYDE-RO Rings are supplied in cartons each containing fifty rings. The packages are light in weight and convenient to handle and store. Rings are manufactured for 4", 6", 8", 10" and 12" cast iron pipe.

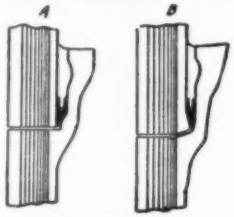


HOW RING IS USED

(1) Stretch the ring over spigot end of pipe with narrow end toward bell.

(2) Slide ring about 1½ inches from end of pipe; then slide the pipe home.

(3) Caulking the ring home, use narrow caulking tool about ⅛" or less in width.



Cross section showing the HYDE-RO Ring caulked into position on plain end (A) and beaded end (B).

HYDE-RO Rings can be used in conjunction with *Tegul*-MINERALEAD, lead or Portland Cement.

Tegul-AMPCO

For Bell & Spigot Terra Cotta and Concrete Sewer Pipe

Semi-rigid, sulphur bases makes tight, root-proof joints. Fills annular spaces completely to secure excellent joint. Reduces infiltration of ground water. Unaffected by dampness, frost or silt coating on pipe. Easily stored and handled. Immune to moisture and weather conditions. 5 lb. ingots; 50 lbs. to carton.

TOTAL LENGTH OF SERVICE

STANDARD DESIGN FOR WATER SERVICES

By MARSDEN C. SMITH

Chief Engr., Dept. of Public Utilities, Richmond, Va.

WHAT is the standard pressure, and what size service does the local water utility require? The tables and figures of this article have been proved by several years' use in at least one large city to be a relatively simple and highly successful means of determining the proper size of water services for most practical conditions.

Basis of Design

The basis of design is a pressure loss between the main and the most remote outlet of 11.3 psi.; of which the service and meter are allocated 7.0 psi. Assuming the water system to be operated so that a minimum of 35 psi. at street level is maintained at all points in the street mains; then this design will provide an outlet flowing pressure of 4.3 psi. (10 ft.) at an elevation of 44.7 ft. above the street level.

This is always adequate for the normal three-story building. But even where pressures in the main are greater than 35 psi. at street level, it is usually unwise to design services smaller than those required by these tables and graphs.

The "load factor multiplier" is used to take into account the fact that not all outlets being served are used at the same time. Of course, the laws of probability recognize that such factors will be exceeded. Yet experience proves these to be remarkably accurate in that the services are of a reasonable size and yet the frequency of overload is so low that no service so designed has had to be enlarged.

Meters

For services of normal length it is less expensive to reduce friction loss by enlarging the pipe than by enlarging the meter. For this reason, meters are rarely used in pipes of their nominal size.

Of course, it must be remembered that an overloaded meter is subject to excessive wear. For this reason, except in unusual applications, the meter should be of such size that the quantity of water found by the tables to be the estimated flow should not cause a greater loss than 5 psi. through the meter. For example, a 3-inch compound meter should not be used for flows in excess of 190 gpm., unless it is known that more than that quantity will be required only at rare intervals. The entire 7 psi. friction loss allocated to both the meter and pipe will be consumed in this meter alone at 217 gpm. flow.

Service Length

Since the pressure loss in a service is a function of the distance that water must be carried in the service, as well as of flow, the length of each service must be known if a correct design is to be made. The distance used in these tables is from the main in the street to the first "riser" from the service pipe. (Sill cocks are not considered to be a "riser".) The risers are then designed to permit a friction loss of not more than 4.3 psi. from the service pipe to the most remote outlet.

Method of Use of Tables

The use of the tables for residential services is simply a matter of knowing the equipment to be served and the service length, and then selecting the proper combination of meter and pipe size from Table I or Table II depending on whether tank toilets or flushometers are used. For a residential service whose exact equipment cannot be found in the tables, interpolations can, as a rule, be readily made.

TABLE I

Meter and Pipe Sizes for Residences Using Tank Toilets

Group No.	Fixtures Proposed or Installed	Size of Service*	Length Range	Size of Service*	Length Range
		in.	*ft.*	*in.*	*ft.*
1.	1 toilet, 1 hydrant	⅝ × ¾	to 225		
2.	1 toilet, 1 kitchen sink	⅝ × ¾	to 105	⅝ × 1	106–280
3.	1 bath, 1 kitchen sink	⅝ × ¾	to 95	⅝ × 1	96–190
4.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks†	⅝ × ¾	to 95	⅝ × 1	96–170
5.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet	⅝ × ¾	to 45	⅝ × 1	46–150
6.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks	⅝ × ¾	to 45	⅝ × 1	46–130
7.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet	⅝ × 1	to 110	⅝ × 1¼	111–300
8.	3 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks	⅝ × 1	to 95	⅝ × 1¼	96–275
9.	3 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	⅝ × 1	to 90	⅝ × 1¼	91–185
10.	3 baths, 2 kitchen sinks, 1 furnace, 1 or more sill cocks, 1 laundry tub	⅝ × 1	to 55	⅝ × 1¼	56–100
11.	3 baths, 2 kitchen sinks, 1 furnace, 1 or more sill cocks, 2 laundry tubs	⅝ × 1¼	to 50	1 × 1¼	51–195
12.	3 baths, 2 kitchen sinks, 1 furnace, 1 or more sill cocks, 2 laundry tubs, 1 extra lavatory and toilet	1 × 1¼	to 85	1 × 1½	86–175
13.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	⅝ × ¾	to 55	⅝ × 1	56–160
14.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet, 1 laundry tub	⅝ × ¾	to 45	⅝ × 1	46–130
15.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	⅝ × ¾	to 45	⅝ × 1	46–130
16.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet	⅝ × 1	to 110	⅝ × 1¼	111–300
17.	3 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	⅝ × 1	to 95	⅝ × 1¼	96–275

*Size of meter used, with larger size service pipe.
†Built-in lawn sprinklers are not included with sill cocks.

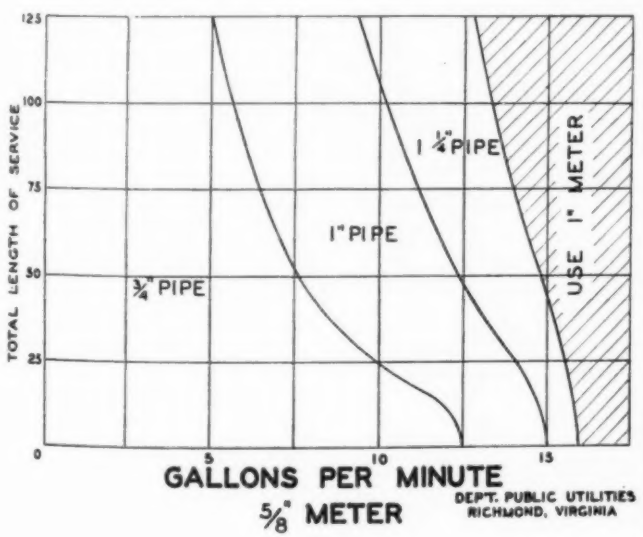


Figure 1

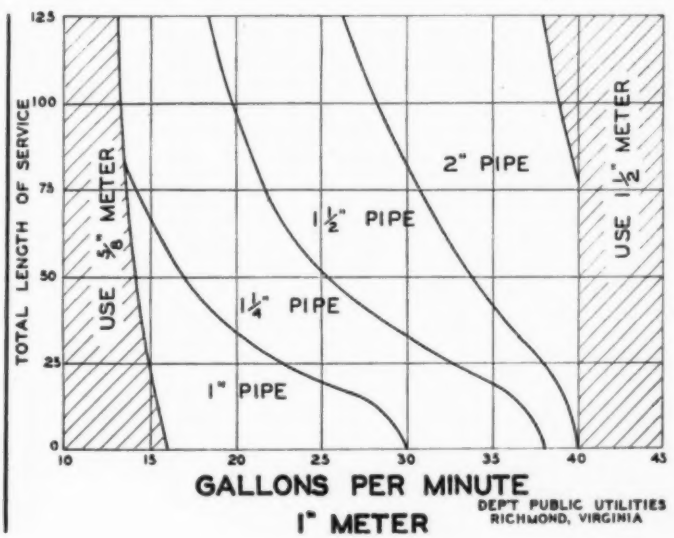


Figure 2

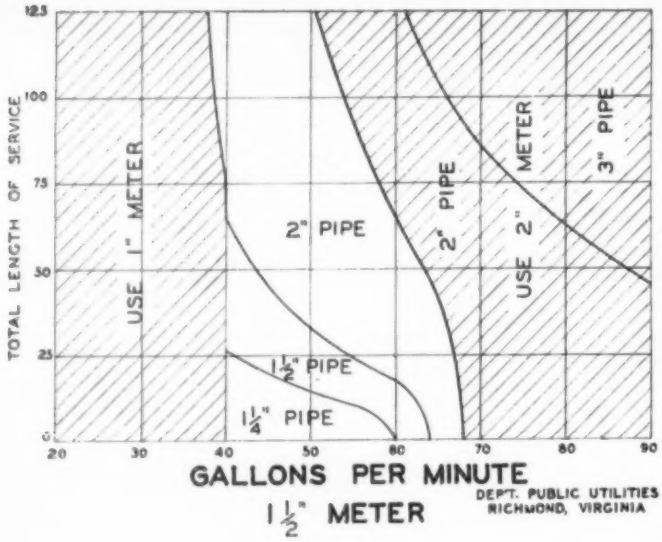


Figure 3

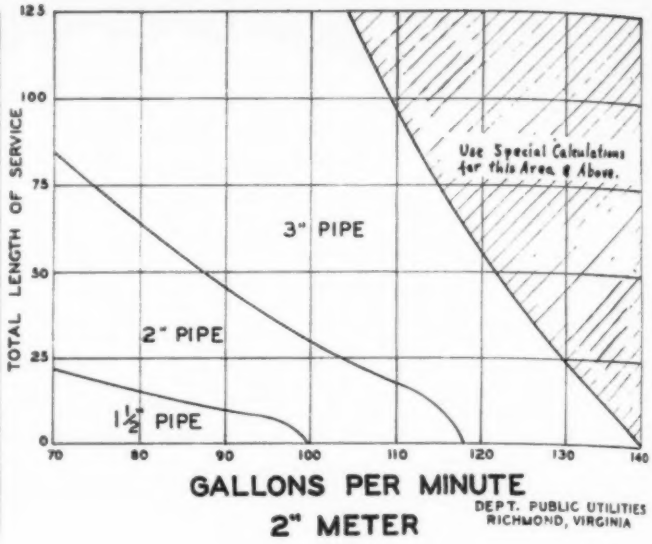


Figure 4

Because of the higher frequency of use of outlets in apartments than in single or two-family premises, a somewhat larger service for apartments (and also for industrial or commercial use) is required. For use in these cases Tables III and IV and Figs. 1, 2, 3, and 4 have been prepared.

The four charts, Figs. 1, 2, 3, 4, show the size of meter to be used for different rates of flow through different lengths of service.

Example 1: An apartment structure contains twelve apartments and each apartment contains a bath with tub equipped with shower, a lavatory and a flushometer toilet, also a kitchen sink. The basement contains two laundry tubs, one slop sink, one furnace, one lavatory, one flushometer toilet, and two sill cocks.

The first step is to determine the total flow required for these fixtures. Referring to Table III the following data are developed.

TABLE III

Demand and Load Factor of Various Appliances

Appliance	Number of outlets	Gal. per minute
Domestic kitchen sinks	2	5
Commercial kitchen sinks	2	7
Shower or shower and tub combined	2	6
Tub or tub and shower combined	2	6
Lavatory	2	4
Tank Toilet	1	4
Bath	5	14
Domestic laundry tubs	1	3
Sill cocks	1 (Max.)	3 (Max.)
Furnace connections	0	0
Steam table sinks	1	6
Soup kettles	2	20
Coffee urns	1	5
Potato peeler	1	8
Dish washer	2	8
Glass washer	2	6
Slop sinks	1	3
Drinking fountains	0	0

The flow of water to Flushometer toilets and urinals should be determined separately from the standard appliances and added thereto in accordance with the following schedule:

No. of Units	Toilet gpm.	Urinal gpm.	No. of Units	Toilet gpm.	Urinal gpm.
1 to 2	30	2	39 to 60	65	7
3 to 4	40	3	61 to 100	70	8
5 to 7	45	4	101 to 141	75	8
8 to 10	50	4	142 to 190	80	9
11 to 15	55	5	191 to 240	85	10
16 to 38	60	6	241 to 400	90	10

TABLE II

Meter and Pipe Sizes for Residences Using Flushometer Toilets

Group No.	Fixtures Proposed or Installed	Size of Service*	Length Range	Size of Service*	Length Range
		in.	*ft.*	*in.*	*ft.*
101.	1 toilet, 1 hydrant	1×1½	to 115	1×2	116–300
102.	1 toilet, 1 kitchen sink	1×1½	to 115	1×2	116–300
103.	1 bath, 1 kitchen sink	1×1½	to 105	1×2	106–280
104.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks†	1×1½	to 105	1×2	106–280
105.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet	1×1½	to 75	1×2	76–145
106.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks	1×1½	to 75	1×2	76–145
107.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet	1½×2	to 200		
108.	3 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks	1½×2	to 200		
109.	3 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	1½×2	to 200		
110.	3 baths, 2 kitchen sinks, 1 furnace, 1 or more sill cocks, 1 laundry tub	1½×2	to 200		
111.	3 baths, 2 kitchen sinks, 1 furnace, 1 or more sill cocks, 2 laundry tubs	1½×2	to 200		
112.	3 baths, 2 kitchen sinks, 1 furnace, 1 or more sill cocks, 2 laundry tubs, 1 extra lavatory and toilet	1½×2	to 160		
113.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	1×1½	to 75	1×2	76–145
114.	1 bath, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet, 1 laundry tub	1×1½	to 75	1×2	76–145
115.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	1×1½	to 75	1×2	76–145
116.	2 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 extra lavatory and toilet	1½×2	to 200		
117.	3 baths, 1 kitchen sink, 1 furnace, 1 or more sill cocks, 1 laundry tub	1½×2	to 200		

*Size of meter used, with larger size service pipe.
†Built-in lawn sprinklers are not included with sill cocks.

Fixtures	Outlets	Flow
12 Tubs	24	72
13 Lavatories	26	52
12 Kitchen sinks	24	60
2 Laundry tubs	2	6
1 Slop sink	1	3
1 Furnace connection	0	0
2 Sill cocks	1	3
	78	196

Now from Table IV, find multiplier for 78 outlets to be 0.28, or 196 × 0.28 = 54.88 gpm., the estimated flow.

From the same Table IV it is found that 13 flushometer toilets require 55.0 gpm. estimated flow. This must be added to the estimated flow of 54.88 gpm. for fixtures, or a total of 109.88 gpm. is to be used for the design.

Now assume that from the main to the first riser is 110 feet; and refer to Figure 4. Here it is seen that a 2-inch meter with 3-inch pipe will deliver 110 gpm. somewhat over 93 feet. If location assures more than 35 psi. in main; that service design would be used even for 110 feet distance here required. But in no event would this distance be increased beyond 121 feet.

Example 2: Industrial Plant. First determine sanitary water flow from Table III:

	Fixture	Flow
20 Showers	40	120
5 Wash Stands @ 6 gpm. each	5	30
30 Toilets (tank)	30	120
20 Urinals @ ½ gpm.	0	10
7 Hose connections	1	3
	76	283

From Table IV, the load factor for 76 fixtures is determined to be 0.28.

$$283 \times .28 = 79.24 \text{ gpm.}$$

Now determine cuisine flow, also from Table III:

3 Soup kettles	6	60
2 Coffee urns	2	10
2 Dish washers	4	16
4 Kitchen sinks	8	28
6 Slop sinks	6	18
10 Drinking fountains	0	0
	26	132

The load factor for 26 fixtures (Table IV) is 0.32.

$$132 \times .32 = 42.24 \text{ gpm.}$$

Now since sanitary flow will not normally be a maximum when industrial and cuisine is a maximum, it is advisable to further reduce this, as long as industrial flow is greater than calculated sanitary flow. And so:

Sanitary flow = 79.24 × .5 =	39.62 gpm.
Cuisine flow =	42.24 "
Industrial flow =	175.00 "
Total design flow	256.86 "

This requires a 4-inch meter and a 4-inch pipe up to 100-foot service length. But a 6-inch pipe would be strongly urged even for a lesser distance than 100 feet.

TABLE IV

Multiply for Load Factor

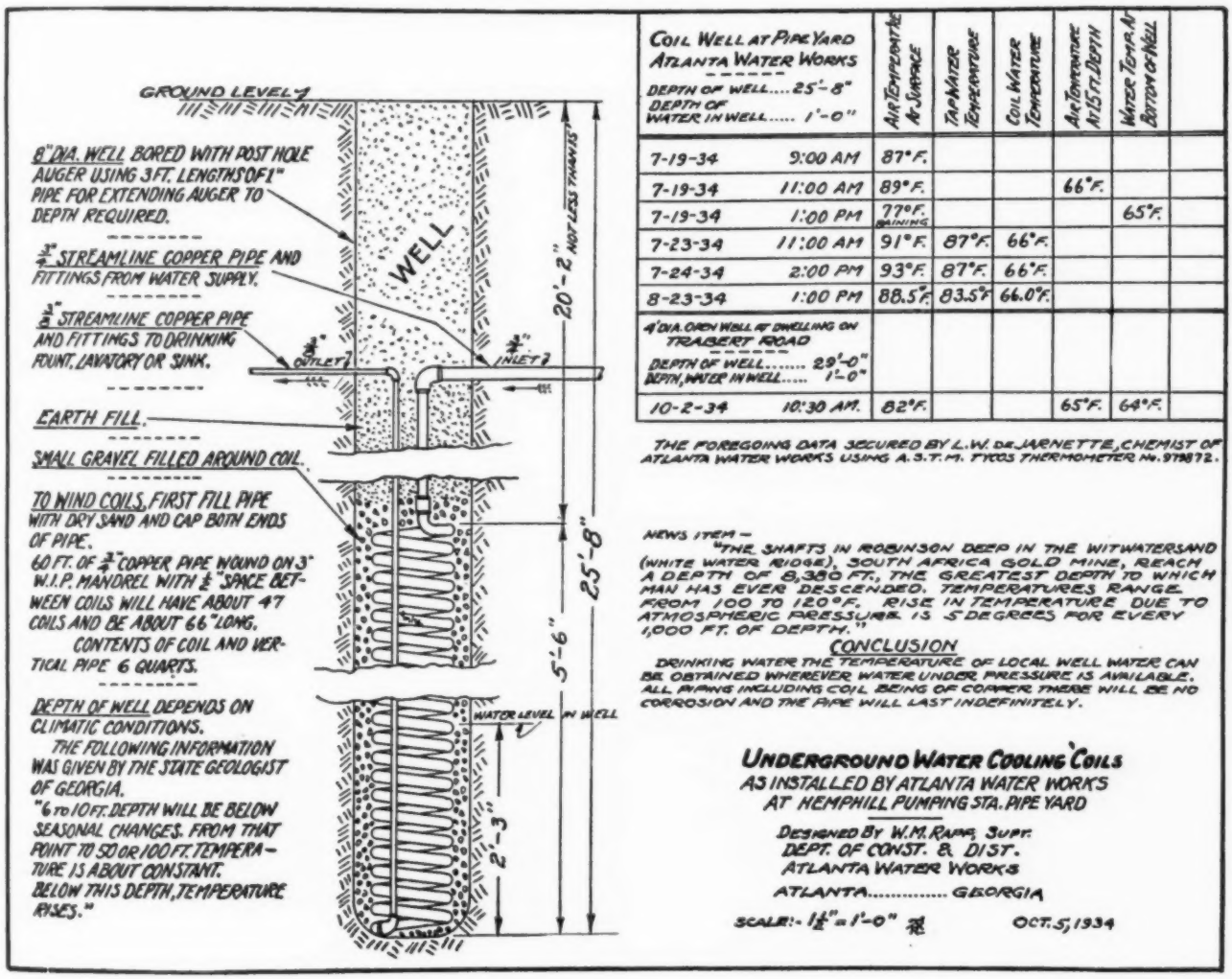
Total No. of outlets	1	2	3	4	5	6	7	8	9
Multiplier	1	.75	.635	.575	.52	.465	.415	.375	.345
10	11	12	13	14	15	16	17	18	19	20
.32	.30	.28	.26	.25	.24	.23	.22	.22	.21	.22
21	22	23	24	25	26	27	28	29	30	32
.22	.24	.26	.28	.30	.32	.33	.33	.34	.34	.34
34	36	38	40	42	44	46	48	50	55	60
.34	.34	.34	.34	.34	.34	.34	.34	.33	.32	.31
65	70	75	80	85	90	95	100	More than 100		
.30	.29	.28	.27	.26	.25	.24	.23	.22		

Notes:

In using these tables and graphs, the following data should be kept in mind.

1) Pipe friction, ¾-inch to 2-inch inclusive C = 130 (copper); 3-inch and above C = 100 (cast iron).
2) Meters used, ⅝ to 2-inch, disc; 3-inch and above, compound.
3) Flow for 5 psig. loss in meter used is ⅝ = 14.5 gpm.; 1-inch = 38 gpm.; 1½-inch = 68 gpm.; 2-inch = 119 gpm.; 3-inch = 188 gpm.; 4-inch = 340 gpm.; 6-inch = 720 gpm.; 8-inch = 1300 gpm.; 10-inch = 1700 gpm.

UNDERGROUND WATER COOLING SCHEME OF ATLANTA WATER DEPARTMENT


GROUND LEVEL
8" DIA. WELL BORED WITH POST HOLE AUGER USING 3 FT. LENGTHS OF 1" PIPE FOR EXTENDING AUGER TO DEPTH REQUIRED.
¾" STREAMLINE COPPER PIPE AND FITTINGS FROM WATER SUPPLY.
¾" STREAMLINE COPPER PIPE AND FITTINGS TO DRINKING FOUNT, LAVATORY OR SINK.
OUTLET
INLET
WELL
20'-2" NOT LESS THAN 15'
EARTH FILL.
SMALL GRAVEL FILLED AROUND COIL.
TO WIND COILS, FIRST FILL PIPE WITH DRY SAND AND CAP BOTH ENDS OF PIPE.
60 FT. OF ¾" COPPER PIPE WOUND ON 3" W.I.P. MANDREL WITH ½" SPACE BETWEEN COILS WILL HAVE ABOUT 47 COILS AND BE ABOUT 66" LONG. CONTENTS OF COIL AND VERTICAL PIPE 6 QUARTS.
DEPTH OF WELL DEPENDS ON CLIMATIC CONDITIONS.
THE FOLLOWING INFORMATION WAS GIVEN BY THE STATE GEOLOGIST OF GEORGIA.
"6 TO 10 FT. DEPTH WILL BE BELOW SEASONAL CHANGES. FROM THAT POINT TO 50 OR 100 FT. TEMPERATURE IS ABOUT CONSTANT. BELOW THIS DEPTH, TEMPERATURE RISES."
5'-6"
25'-8"
WATER LEVEL IN WELL
2'-3"

COIL WELL AT PIPE YARD
ATLANTA WATER WORKS

DEPTH OF WELL..... 25'-8"
DEPTH OF WATER IN WELL..... 1'-0"

		Air Temperature At Surface	Tap Water Temperature	Coil Water Temperature	Air Temperature At 15 Ft. Depth	Water Temp. At Bottom of Well	
7-19-34	9:00 AM	87°F.					
7-19-34	11:00 AM	89°F.			66°F.		
7-19-34	1:00 PM	77°F. RAINING				65°F.	
7-23-34	11:00 AM	91°F.	87°F.	66°F.			
7-24-34	2:00 PM	93°F.	87°F.	66°F.			
8-23-34	1:00 PM	88.5°F.	83.5°F.	66.0°F.			
4" DIA. DRIVEN WELL AT DWELLING ON TRABERT ROAD							
DEPTH OF WELL..... 29'-0" DEPTH, WATER IN WELL..... 1'-0"							
10-2-34	10:30 AM.	82°F.			65°F.	64°F.	

THE FOREGOING DATA SECURED BY L.W. DE JARNETTE, CHEMIST OF ATLANTA WATER WORKS USING A.S.T.M. F105 THERMOMETER No. 979872.

NEWS ITEM —
"THE SHAFTS IN ROBINSON DEEP IN THE WITWATERSAND (WHITE WATER RIDGE), SOUTH AFRICA GOLD MINE, REACH A DEPTH OF 8,380 FT., THE GREATEST DEPTH TO WHICH MAN HAS EVER DESCENDED. TEMPERATURES RANGE FROM 100 TO 120°F. RISE IN TEMPERATURE DUE TO ATMOSPHERIC PRESSURE IS 5 DEGREES FOR EVERY 1,000 FT. OF DEPTH."

CONCLUSION

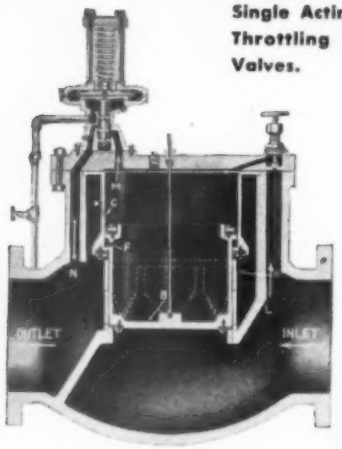
DRINKING WATER THE TEMPERATURE OF LOCAL WELL WATER CAN BE OBTAINED WHEREVER WATER UNDER PRESSURE IS AVAILABLE. ALL PIPING INCLUDING COIL BEING OF COPPER THERE WILL BE NO CORROSION AND THE PIPE WILL LAST INDEFINITELY.

UNDERGROUND WATER COOLING COILS
AS INSTALLED BY ATLANTA WATER WORKS
AT HEMPHILL PUMPING STA. PIPE YARD

DESIGNED BY W.M. RAPP, SUPT.
DEPT. OF CONST. & DIST.
ATLANTA WATER WORKS
ATLANTA..... GEORGIA

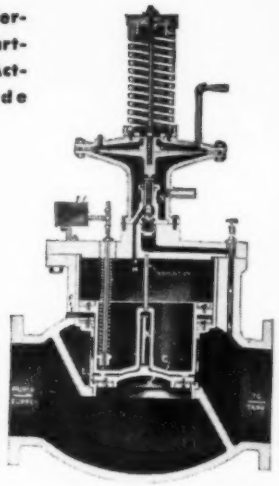
SCALE:- 1½" = 1'-0" OCT. 5, 1934

Golden-Anderson Single Acting Semi-Throttling Altitude Valves.



For New Construction or Rehabilitation

Golden-Anderson Stop Starter Single Acting Altitude Valves.

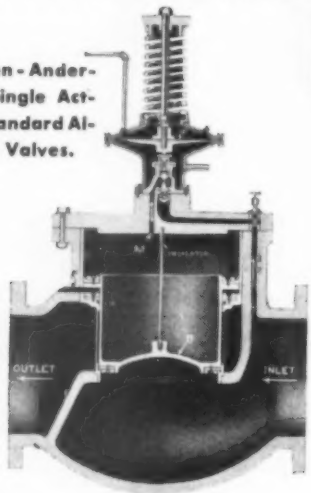


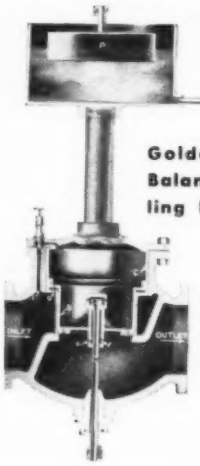
STANDARDIZE ON

GOLDEN-ANDERSON VALVES

for Engineered Protection

Golden-Anderson Single Acting Standard Altitude Valves.





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Golden-Anderson Altitude Control Valves require no floats or complicated fixtures in their installation. They can be easily installed and protected from inclement weather to assure unfailing water service control under any climatic conditions. Sensitive operation is another outstanding feature—they faithfully maintain water levels within 3" to 12" variation and may be arranged with auxiliary equipment to perform a multiplicity of services. Write for latest technical catalog.

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WATER WORKS RULES AND REGULATIONS*

By MELVIN P. HATCHER

Director, Water Dept., Kansas City, Mo.

THE water works industry is striving right now to improve its technique in public relations matters. Procedural awkwardness and lack of consistency in matters involving contacts between the water works properties and their customers are likely to be a serious deterrent to any progress in this direction. Faulty operations may be overcome only if the employees and the management know the answers to customer contact problems. This principle, in turn, means that the answers to the main problems must be written down in black and white.

In brief, this is the argument favoring water works property rules and regulations. Existing rules vary widely in scope, arrangement, formality and the degree of perfection attained. It is likely that the larger properties have greater need for written rules than do the smaller properties. Yet it appears that few of the former have revised their regulations in the last five or ten years. It is obvious that these rules need frequent revision if they are to serve adequately. These considerations, together with the fact that the whole subject seems to deserve more attention than it has been given, have prompted the author to undertake this paper.

It should be said at the outset that the water works property rules and regulations must be regarded as a formal document. They will be referred to in most service contracts, but even if not mentioned, they are likely to be a part of the contract. They may not exist as a document, or, if they do, they may be used formally only on rare occasions. In any event, they are undoubtedly a part of the utility's contract with its customers.

Most rules and regulations include detailed statements of rates for service, billing and collection, meter installation and repair, and other important practices, although, as will be more fully discussed later, the organization or arrangement of these rules follows no generally accepted pattern. Beyond providing thoroughly for some of the more important practices, a varied amount of attention is given to minor items which can be rather troublesome and with which the water works management in every locality is called upon to deal from time to time, such as the problems involved in the prohibition against services laid across one private property to reach another, the prohibition against more than one service into any one building and similar matters that tend to add to the difficulty of orderly administration.

As a class, these problems are related in one way or another to the arrangement of services and meters and, in a lesser degree, to interior piping on the customer's premises. It may not be extremely important whether they are covered in a formal document, but it is necessary that the utility have a solution for each of them which will consider the customer's point of view. There is little in water works literature to indicate industry-wide concern with these problems. Most properties have evolved their own practices out of their individual experience. Some excellent advice concerning interior piping has been provided in the handbook, "Water-Supply Piping for the Plumbing System," prepared by the National Association of Master Plumbers[1]. No study of equal competence has been made of the many difficulties with the arrangement of services and meters. *Water Works Engineering* has rendered excellent service to the industry through its "Question and Answer" section on operating problems, but there are quite a few rules and regulations problems that cannot easily be disposed of in this way. They need more generous treatment with illustrations and reports on the effect of some of the bad practices.

The author sought answers to a number of the questions just described, as well as information on the whole subject of regulations, from about fifteen of the larger water works properties. Of these, only two replied that they had no printed rules and regulations, and nearly all responses included some interesting comments on the need for them. The material to follow comprises a digest of the questionnaire replies and the author's observations on some of the problems that have been encountered in Kansas City in the last six or seven years.

Value of Written Rules

Is a written set of rules and regulations essential to the orderly administration of matters involved in the contacts of the water works properties with their customers? Is such a set of rules necessary for a proper understanding between the utility and its customers? Should not each water works utility provide itself with rules and regulations in document form in order to have them available for reference against complaints about any practice or for use as evidence in any court action questioning the utility's authority? Finally, would not such rules be a means of insuring employee acquaintance with practices and procedures?

None of the fifteen water works managers who replied to the questionnaire was willing

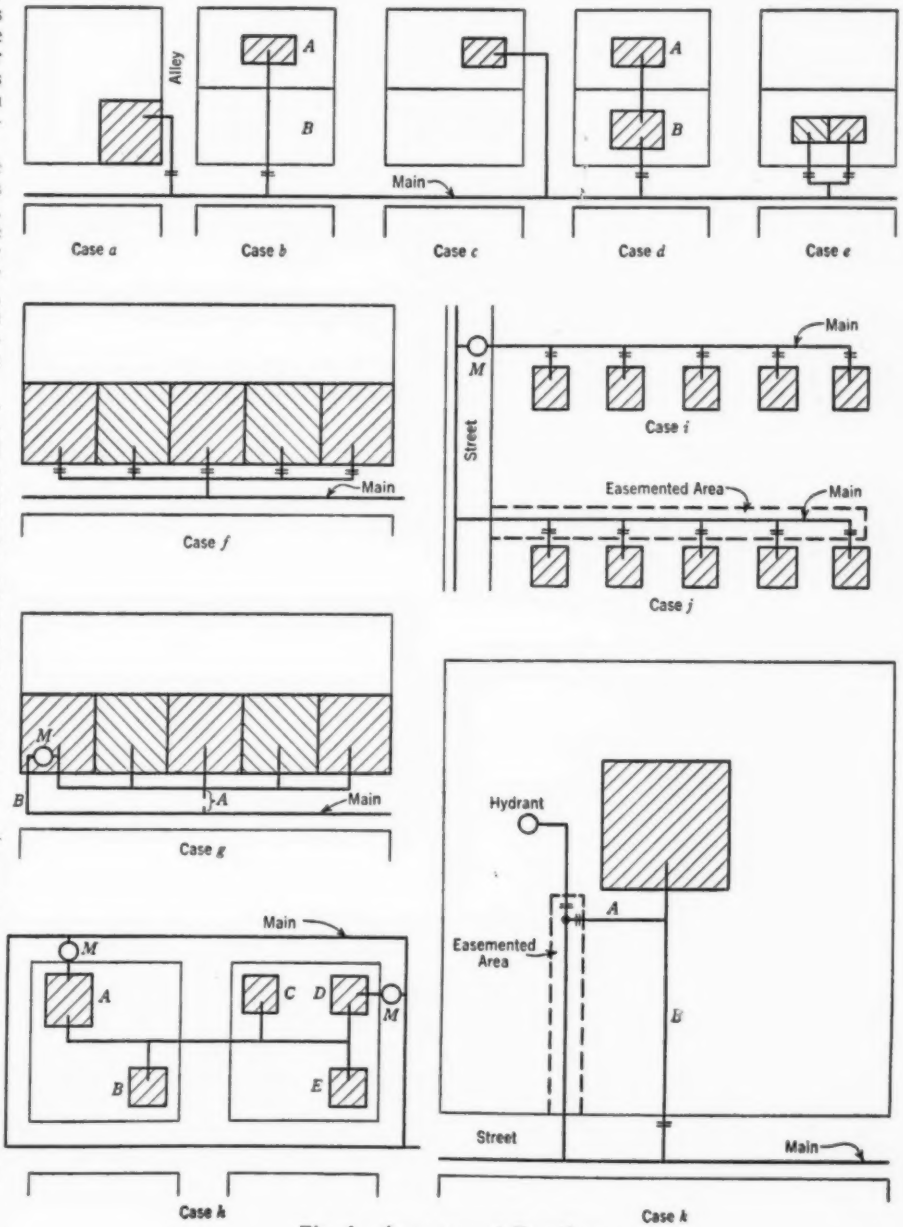

Fig. 1. Arrangement Practices

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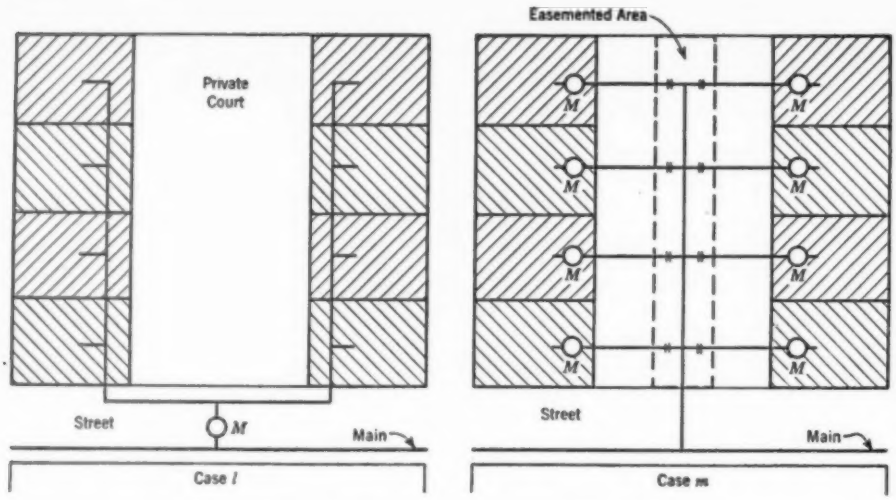


Fig. 2. Effect of Change in Type of Ownership.

to deny that written rules were necessary, although two indicated that they did not have them. One said he had found little need for a complete re-compilation of rules last assembled in 1927, and a few others stated that they were using rules which had not been brought up to date in the last ten years. Nearly all of the replies stressed the need for rules, citing one or more of the reasons given in the paragraph above.

In the author's opinion, written rules and regulations are important tools for the operation of a water works utility, although the degree of their value is probably related to the size of the property. The requirements of a smaller property may be very well satisfied by a set of rates and unassembled rules on such fundamental matters as main extensions and meter and service ownership and maintenance. As the number of operations increases with size, it becomes more and more necessary for management to establish rules and regulations which will provide a ready answer for all except a small percentage of the customer contact problems that arise.

First things must come first, however, and there is always plenty of administrative detail to demand the attention of management. It must be admitted that most properties have not found it necessary to place major emphasis on rules and regulations. Many apparently operate with rules that have been outgrown and are overrun with practices established by precedent or administrative order. Probably the point to be stressed is that the compiling of rules and regulations requires a study and review of all utility-customer contact problems in addition to those involved in the installation of mains, services and meters. The rules will have achieved one of their main purposes if their compilation and continued revision have caused management to make the necessary studies of all of these practices.

Scope and Purpose

What should be the scope and purpose of the rules? Into how much detail should they go in defining the utility's and the customer's respective rights and obligations?

The main purposes of rules and regulations are quite clear: to complete the service contract with customers, to set forth the conditions of service, to state the utility's obligation to the customer and the reverse, and to cite the utility's authority for its prohibitions and requirements. Their scope is not so readily defined. Rates, billing practices, service and meter ownership and maintenance, discontinuance and restoration of service, access to premises, and matters of similar basic import are included in nearly all rules and regulations. Considerable differences occur in the manner in which various utilities deal with interior piping on the customer's premises and in the treatment and the amount of attention given to provisions for the orderly and administrable arrangement of mains, services and meters. Those parts of the rules and regulations relating to items in the latter category deserve, in a way, to be set apart from the fundamental matters referred to above. They are less important in the sense that they may very properly be covered in administrative orders, with only the authority for such orders written into the main body of the regulations.

The term "main body" is meant to apply to that part of the rules and regulations which is passed on formally by some ruling authority; the portion fixed by ordinance, for example, in municipally owned water works properties, or the part approved by a public utilities commission in some privately owned utilities. Setting the minor elements apart from the main body makes it unnecessary for changes in the former to have the approval of the governing authority.

A description of the plan for Kansas City's rules will help to explain the point just made. The main controlling ordinance covers only the basic subjects previously noted, such as rates and billing practices, but it includes the provision that the director may make rules regarding the arrangement of meters and services and may fix the prices of work done for customers or others. The director's rules, in turn, refer to standard drawings for services and meter settings, thus overcoming the need for verbal descriptions of these facilities and the materials going into them. Placing these matters apart from the main ordinance gives them a desirable degree of flexibility.

Water works rules may exercise direct control over whatever aspect of interior piping on the customer's premises concerns the water supplier: cross-connections, back siphonage, and temperature and pressure relief devices. Alternatively, this control may be provided for in a building code administered by the city, for privately owned properties, or by a city department (generally, the public works department) other than that of water supply, for a municipally owned property.

Many questions are raised about the minuteness of detail with which practices dealing with interior piping or the arrangement of meters and services should be described. To meet these problems, each water works property should have clearly defined solutions which must be thoroughly understood and consistently applied by its employees. The final section of this paper deals extensively with these practices.

Standardization

Would standardized rules and regulations be an advantage to the water works industry?

The questionnaire replies agreed that the standardization of rules and regulations is impracticable because of the wide range of conditions encountered. At least two replies, however, approved the idea of "model" regulations. This suggestion has had at least one trial. The Maine Water Utilities Assn., in co-operation with the Maine Public Utilities Commission, has worked out model but flexible rules that apparently have served the purpose well.

Nevertheless, the author doubts that model rules could be made to justify the time and effort needed to create them, except for use by closely knit groups of properties having common problems. It appears that the most good could come from concerted effort if it were aimed at an agreement on an outline of the various procedures and a summary of opinions on specific practices based on experience with them.

Rules and regulations have certain logical subdivisions:

1. Fundamental matters such as rates, billing practices, service and meter ownership and maintenance, discontinuance and restoration of service, access to premises, misuse of hydrants, penalties for violations and the like.
2. Standard practice rules relating to the arrangement of mains, services and meters.
3. Prices on work performed for the customer or for others.
4. Water main extension rules.

In many rules all of these parts are commingled, although the fourth is more often treated separately. From the standpoint of their usefulness to the management that has prepared them, this is not necessarily a disadvantage, because the management knows, or should know, its own rules thoroughly. But water works managers seeking to use the rules of other properties as a guide to the formulation or revision of their own regulations could make more intelligent use of those that adhered to a more or less standard outline. Certainly, more progress would be made in the perfection of rules if there were less dissimilarity of outline.

The complexity of rules and regulations stems largely from confusing the first three subdivisions. If these are followed strictly, the subject becomes simplified. The first subdivision lends itself to arrangement under topical headings set up in logical sequence. The second can be brought together under three or four main headings. This outline plan will shorten the time needed to comprehend the rules. The reader will find it easier to digest them if he can discern the main outline and can feel that the material on any one subject is under one heading.

The force of this statement will be apparent to anyone reading a set of rules and regulations with the proposed separation in mind. Much of the difference in rules and in the range of items covered is centered in the second group. This subdivision has to do with the number of services that will be run to multiple-family dwellings or commercial buildings, the prohibition against the

extension of services from one property to another and so forth. If the material in this category is set aside, the rules of the various water works properties will be found to differ to a much lesser degree.

TABLE I

Topical Headings for Kansas City Rules

1. Water Funds
   - *a.* Water Fund
   - *b.* Water Debt and Interest Fund
   - *c.* Water Deposit Fund
2. Service Availability
3. Unauthorized Use of Water of Facilities
4. Water Rates
   - *a.* General Meter Rates
   - *b.* Suburban Meter Rates
   - *c.* Flow Line Rates
   - *d.* Temporary Service Rates
   - *e.* Domestic Flat Rates
   - *f.* Fire Protection Rates
5. Meter Reading and Building Practices
6. Water Temporarily Unmetered
7. Bill Payment Guarantees
8. Service Connections to Mains
9. Water Service Maintenance
10. Water Service Disconnection
11. Water Meters
12. Access to Premises Served
13. Service Discontinuance
14. Water Hydrants
15. Water to Other City Departments
16. Misuse of Property
17. Price Schedules
18. Rules and Regulations
19. Penalty for Violations

To indicate that the first subdivision can easily be arranged by topic, the headings for this part of Kansas City's rules are listed in Table I.

This outline aims at directness, logic and simplicity. None of the subject matter under the headings is burdened with the detail required for main, meter and service arrangements or for prices of work done, which are included in the second and third subdivisions previously described. These items are dealt with separately under the authority granted by "17. Price Schedules" and "18. Rules and Regulations" in the main controlling ordinance.

The conclusion, then, to the query, "Would standardized rules and regulations be an advantage to the water works industry?" is summarized in this way: Standardization is not a solution. The industry might profit from the formulation of model rules but just as much good would probably result from adherence to an outline coupled with more industry-wide studies of procedures for the arrangement of mains, services and meters.

Arrangement Practices

The remainder of this paper is devoted to a digest of replies to the questionnaire and to observations by the author on the arrangement of mains, services and meters. The list of problems and practices is not complete; almost any water works manager could think of others that he has encountered. The summary below will, however, serve to show the relationship of this part to the entire rules and regulations problem.

Services or private mains in alleys (Fig. 1, Case *a*). The question is whether it is wise or proper to allow services or mains to be laid in alleys. Permission to make a short extension into an alley, as shown in the drawing, could lead to extensions of greater length. In that event, the service takes on the character of a private main. Mains or services in alleys are difficult of access. Only a few of those replying approved the practice.

Service across private property (Fig. 1, Case *b*). Services across private property can be productive of trouble. Owner *A* could acquire the property without knowledge of the water service situation, and Owner *B* could deprive Owner *A* of water service unless the right-of-way across Owner *B*'s property were fixed by easement and recorded. Moreover, if there were no entry in the records revealing this irregular arrangement during a routine turn-on or turn-off of the water, the serviceman would have difficulty locating the curb stop serving Owner *A*. About one third of the questionnaire replies approved the practice, but then only if an easement is obtained.

Service along street axis (Fig. 1, Case *c*). Most rules prohibit this practice by providing that the service shall be connected to a main in the street abutting the premises to be served, within the limits of the property's footage on that street. It is also well to provide that the service from the main to the curb stop shall be laid normal to the main's axis. Approximately one fourth of the questionnaire replies, however, indicated approval of the practice. It appears that exceptions should be allowed where it is clear that no other customers will seek connection to the extended service.

More than one customer per service (Fig. 1, Case *d*). The dangers of having more than one customer per service are obvious. Owner *A* is wholly without rights of service and there is scarcely a way that they can be guaranteed to him. Premises *A* can be sold without the purchaser's knowledge of the service location. In a disagreement, both owners (or all the owners, if the service is extended to additional premises) will expect the utility to settle their problem. Only one reply indicated approval.

Double-header services (Fig. 1, Case *e*). Kansas City, with customer ownership of the service from the house to the main, has administered many double-header service installations with fair success, although in recent years only two-family residences have been permitted to run them. If there is failure to agree on joint replacement from the curb stop to the main, either or both customers are required to make new separate attachments. Kansas City has recently agreed to carry out all service maintenance from the main to the curb stop. Under this arrangement, the double-head service is encouraged, though it is necessary to limit the distance between curb stops (now 15 ft.) and to fix the relation between the distance from the curb stop to the building entrance and the distance from the building entrance to the main. If the distance is too great, the customer may ask for a new main-to-curb service when his own portion needs replacement. All but two of the questionnaire replies approved this practice.

Joint service branch to commercial buildings (Fig. 1, Case *f*). A joint service arrangement for commercial buildings near each other is more economical than separate services with two main attachments. Moreover, there is some advantage to the water works property, particularly if it is municipally owned, because the branch service reduces the number of openings in the pavement. If the practice is permitted, it is almost essential that the utility assume the responsibility for maintenance from the main to the curb stops. In addition, the utility should reserve the right to have attachments made on either end of the branch service. This brings it into the private main category and makes the whole practice a variant of the two main plan. Only about one third of the questionnaire replies favored the idea. The author believes that thorough study of the practice would lead to its universal adoption, since it results in economies and serves a useful purpose.

An interesting sidelight on this practice is shown in Fig. 1, Case *g*. A person owning all the buildings served effected some savings in the cost of water by purchasing it through one meter at *M* over a new service marked *B*. The owner then connected the old branch service into his own distributing header and disconnected the old service at *A*. This procedure is of course wholly improper because of the use it makes of facilities on public property, but it does illustrate the need for rules that will anticipate and guard against such unusual practices.

Multiple metering (Fig. 1, Case *h*). *Should a single industry or institution owning buildings A, B, C, D, E be permitted to take water through more than one meter and service? Is it advisable to treat combined meter readings for billing purposes as though the water were delivered through a single meter? Should the industry be permitted to carry its own mains across city streets to effect this consolidation?*

These questions concern only a small number of the larger industrial or institutional customers and few rules and regulations deal with them adequately. The use of mains on more than one street or in more than one block is preferred by industries for whom reliability of service is important. Assuming that the rates for service are designed to assess the extra costs involved equitably, multiple metering is objectionable to the utility. Changes in the quantities of water taken through the meters will make it extremely difficult to detect faulty meter registration.

Kansas City does not permit the practice in new installations. It has had experience with multiple-metering problems growing mainly out of the vacating of streets for certain sizable industries. It overcame one large industrial customer's complaint against service from a single main by allowing a battery meter setting with connections to mains on two streets. Questionnaire replies, however, approved the practice by a ratio of three to two.

Should the meter readings be combined for billing purposes? This, of course, has more to do with rates than it does with rules and regulations. Actually, under rate plans that do not provide for service charges, meter readings should be combined only if the meters are set in battery. The author favors a service charge on large meters as a means of curbing multiple metering and claims for combined meter readings.

Should the industrial customer be permitted to extend mains across city streets to effect the inter-connection of properties in two or more blocks? The questionnaire replies were about evenly divided on this point. Kansas City has allowed one such inter-connection, and the water main was laid in a tunnel under the street.

Purchase of water for resale (Fig. 1, Case *i*). The purchase of water through a master meter on public property, the distribution of the water through mains on private property and its resale on a metered or flat rate basis to individual customers is a procedure which the author doubts should be condoned at all, although about half of the questionnaire replies indicated approval. The owner of the distributing mains can withdraw from participation in the resale plan at any time. Property served can change hands without the purchaser's knowledge of the water service condition. Individual customers can be deprived of their water supply through no fault of their own and may find themselves without adequate means of correcting the trouble.

Present Kansas City rules prohibit the practice. A few such installations were made about twenty years ago, including one system installed by a real estate firm to serve about 100 homes in a low-value district. Recently, as the wrought-steel mains began to fail, the owner sought relief and finally gave notice of his intent to withdraw his guarantee of bill payment. It then became necessary for the city to install mains in city streets to serve the system's customers.

Utility ownership of mains on private property (Fig. 1, Case *j*). Utility ownership of mains on private property is a procedure which assumes that the utility will own and operate the mains in very much the same way that it operates them in city streets. The utility must therefore have a perpetual easement giving it the right to own, install, operate and maintain the main without liability for property damage of any kind. Consequently, the property owner must not be permitted to place buildings or other obstructions on the right-of-way.

The problem is one of orderly administration. It appears clear in a legal sense that the utility could make contracts for service with individual owners and could discontinue it by operating a curb stop, just as it does where the curb stop is on public property. Periodic inspections would be needed to maintain right-of-way accessibility. Should the owner violate the easement, litigation might be required, tending to create a difficult public relations problem. Approximately one third of the questionnaire replies approved the practice. It is the author's opinion that the practice should rarely be permitted.

The matter of easements bears on another very important problem, illustrated in Fig. 1, Case *k*, which arises with much greater frequency than the preceding case. Customers find it difficult to understand why commercial services should not be connected to fire protection services on private property. This practice was not included in the questionnaire but the author has the impression that exceptions are seldom made to the rule that each of the two classes of service must have its own provision for shut-off on public property. Each service has its own contract; failure to pay the bill on one should not affect the other. A commercial connection to a fire protection service makes it impossible to use the detector meter on the latter. It is not possible to meter both services on the same meter in a satisfactory way. This situation could be overcome by easements containing the provisions previously described, but again they would create conditions practically impossible to administer. The author would favor this type of easement only very rarely.

Kansas City has permitted one such easement to solve another type of problem. A multiple-family dwelling was originally built for single ownership with a single service and meter as shown (Fig. 2, Case *l*). Later the site was subdivided and the apartments were sold to a number of individual owners, each requiring a separate contract for water service. An easement was obtained from every owner for a water main across his portion of the court separating the two apartment structures (Fig. 2, Case *m*).

To emphasize the point that much is left unanswered in rules practices, the final problem presented concerns the limitations on the number of services per building. *Should each of the families in a two-family dwelling be allowed a separate service and be required to make individual contracts? What of four- and six-family dwellings?* Individual services to tenants on two or more floors of a commercial building pose a similar question. The author's present opinion is that the limitations are fixed by the number of stops that can be handled at the curb without too great a risk of confusion. Two services to a building are enough. Kansas City has one ten-family dwelling that insisted years ago on individual services, in which it is necessary to disentangle the inter-connections periodically. Buildings having more than two tenants should be required to make their own apportionments of the cost of water service.

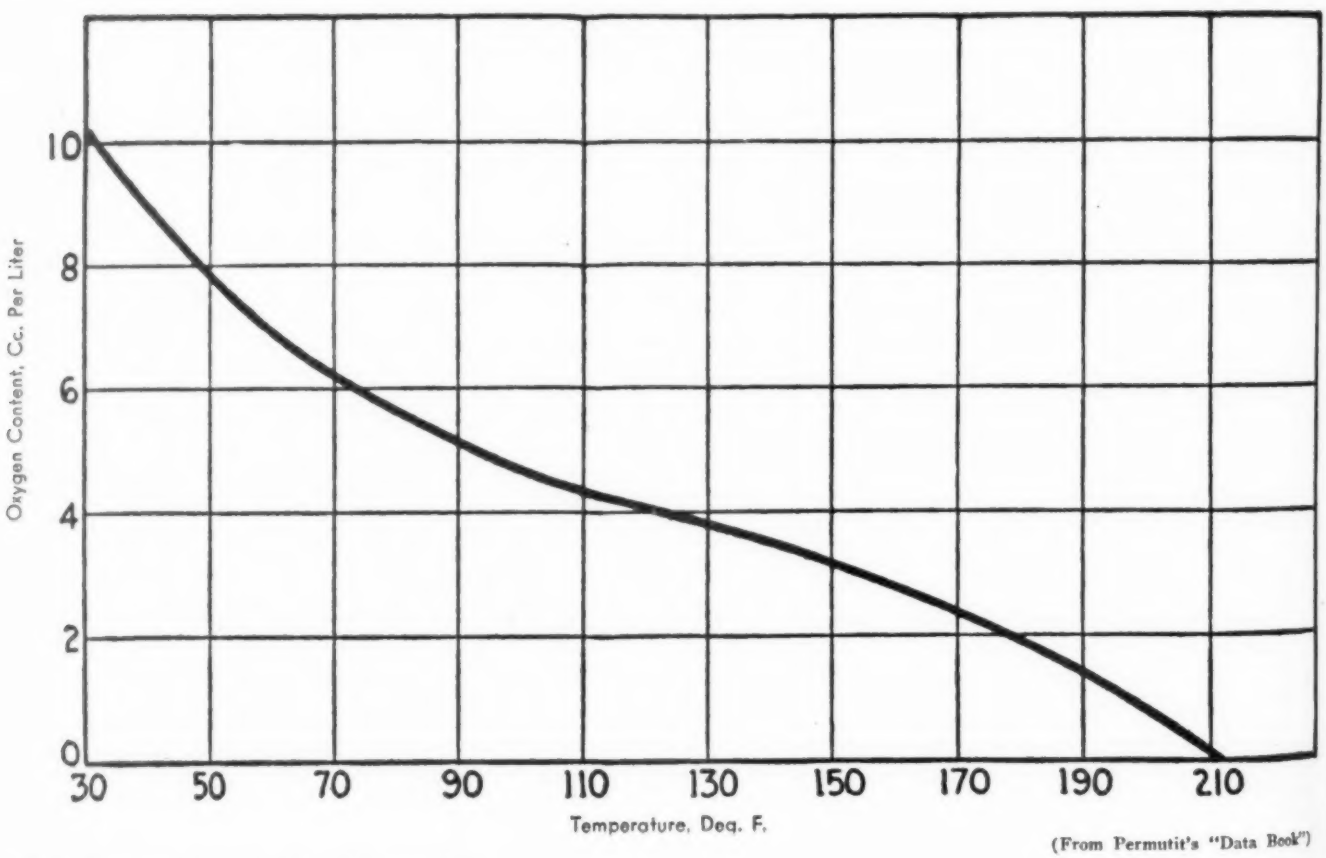
In conclusion, it should be observed that there is more lack of uniformity than there need be in the rules bearing on many of these practices and that it surely would profit the industry to agree on many of them.

Reference

1. Dawson, F. W. & Kalinske, A. A. "Water-Supply Piping for the Plumbing System." Natl. Assn. of Master Plumbers, Albany, N.Y. (1942).

SOLUBILITY OF OXYGEN IN PURE WATER

At Various Temperatures



(From Permutit's "Data Book")

COMMUTATOR MAINTENANCE

By C. B. HATHAWAY
Westinghouse Electric & Mfg. Co., East Pittsburgh, Pa.

COMMUTATOR maintenance of industrial size motors can be made easier when an understanding exists of what goes on in the motor. A commutator, being an assembly of many parts, is only good as long as all of the parts stay in the same mechanical location with respect to each other. If one or two bars move out of place with respect to the others, the commutator becomes rough and sparking will occur when they pass under the brushes.

Construction of Commutators

Practically all commutators used in industrial machinery in sizes above 1 hp. are of the vee type. In such a commutator, a vee-shaped groove is machined in each end of the copper commutator bars and separating mica segments. A corresponding inverted vee is machined at the back end of the commutator bushing and in a separate retaining ring that assembles at the other end of the bars. The bars are clamped between the two vee rings and are insulated from them by a molded mica vee ring.

The machining of the vee ring on the bushing and steel retaining ring must bear

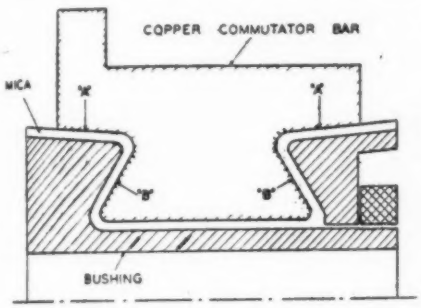


FIGURE 1
Fig. 1.—Cross-Section of Commutator Showing Vee Type Construction

a definite relation in radius to the machining of the vee grooves in the bars. Before assembly, the steel vee rings have a slightly smaller diameter than the vee grooves in the bars. When assembled, the pressure applied to the steel retaining vee ring draws down on the inner edge of the vee groove of the commutator bars as indicated by arrows marked "B" in Fig. 1. This reduces the diameter of the assembled segments and thereby places them in arch binding.

It is also important to have the upper edge of the vee groove, as indicated by arrows marked "A" in Fig. 1, make firm contact with the mica vee rings and the commutator steel parts. This provides a seal against the entrance of dirt at the edge of the bars and also puts an outward stress against the tips of the bars. With this initial stress in the ends of the bars they are less likely to be thrown outward due to centrifugal force when the commutator rotates. The ends of the bars will not move until the outward stress due to the centrifugal force exceeds the initial stress in the bars.

The necessary elasticity to compensate for expansion and contraction in the commutator resulting from different temperatures is provided in the mica. The strain placed upon the mica segments in compression must be such that the bars are held firmly together when the commutator is cold and must be within the elastic limit of the mica when the commutator is hot. The rise in temperature of the commutator is, of course, accompanied by the expansion of the copper in all directions. The chief concern is with the expansion that occurs in the bars circumferentially. If the commutator has especially long bars, some provision must be made to absorb the expansion along the length of the bar.

Both the mica segments and the mica vee rings are made up from flake mica which has a diameter of about 1¼ in. The flakes are held together with shellac. The built-up mica parts are thoroughly pressed and baked before they are assembled with the other parts of the commutator.

All commutators should be seasoned after assembly with new mica parts. The purpose of the seasoning is to drive out all excess shellac at the pressure points. This can be done with one heating and tightening but a much better job results when the operation is repeated several times.

An oven in which commutators are seasoned should be held at a temperature between 160° and 180° C. The temperature of the commutator should not be permitted to exceed 150° C. or slippage of the mica flakes will occur when pressure is applied. These mica flakes will come out above the surface of the bars. There is no danger of annealing the copper which has been cold worked to obtain hardness of these temperatures. Cold worked copper will not anneal until temperatures above 200° C. are encountered.

Commutation—A Maintenance Problem

Commutation is a maintenance problem because of the damage done to the commutator by the brushes. Sparking at the brushes, if at all severe, results in burning the surface of the commutator and causing the brushes to ride poorly. The effect is cumulative and continues until the sparking becomes so bad a flashover may occur, and as a result the machine must be withdrawn from service and repaired.

The sparks occurring at the brush result when the current is reversed in a coil as the bars, to which its leads are attached, pass under the brush. The time of this reversal is extremely short and therefore high reactance voltages are set up. To overcome the adverse effects of these high voltages, a commutating pole or interpole is frequently used to set up a voltage from the coil in which the current is being reversed to counteract this reactance voltage. If it were possible to completely compensate for this reactance voltage, no sparking would occur at the brushes, providing the commutator is smooth and the brushes ride evenly.

Occasionally every third bar around the commutator shows smut while the two bars in between have a normal color. This can happen where there are three bars per slot. In the case where there are four bars per slot, every fourth bar is sometimes found blackened. This is due to improper strength of the commutating field or improper shape of the face of the commutating pole.

The commutating condition for each of the three conductors in the slot is different. The energy set up by the reversing of the current in coil "A" in Fig. 2 is partially transferred to the coil "B" by mutual inductance. When coil "B" starts its commutation it has a chance of unloading part of its energy to coil "C," but when coil "C" is commutated there is no place to deliver its energy; therefore it is more likely to deliver it as an impulse through the brushes. This impulse, of course, may show up as a spark or an arc of very short duration.

Since the commutating condition in the three coils is different, the commutating pole design can only be made to take care of the average condition and the inherent energy absorbing capacity of the brushes must be relied upon to take up the difference between the average and the peak values of voltage. If the strength of the interpole is not adjusted for this average, the strength of the peak energy is apt to be greater than can be absorbed by the brush. Obviously, this peak will be excessively strong in only one coil of the group in the slot. Such a condition can be eliminated by correcting the strength of the interpole flux.

Carbon Brushes

An inherent characteristic of carbon

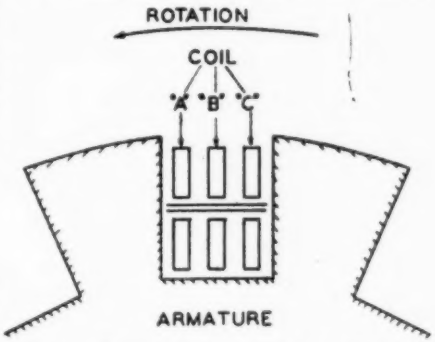


Fig. 2.—Cross-Section of Rotor Showing Coil Arrangement in Slot.

brushes that ride upon a moving commutator is the drop of one volt between the face of the brush and the surface of the commutator. This voltage drop varies slightly with current, but for all practical purposes it may be considered as one volt. Some brushes have much higher contact drop than this, while others, such as the copper-graphite brushes, have much lower contact drop, but these will not be considered here as they are used in very special applications. As mentioned before, this one-volt drop provides a cushion to absorb the energy set up by the armature coils being commutated—that is, not fully neutralized by the commutating poles.

The density of the current in the brushes does not follow a hard and fast rule. Modern electro-graphitic brushes will operate satisfactorily with densities up to 65 amperes per square inch continuously. However, accepted practice holds this density to between 40 and 50 amperes per square inch. Many cases have arisen where responsibility for poor commutation is blamed on high density and additional brushes have been insisted upon. There is no evidence to support the contention that an increased number of brushes, when applied only for the sake of reducing the current density, has been beneficial. Additional brushes may, however, improve the mechanical contact problem and thereby be beneficial. For example, if a motor has but two brush arms with only one brush per arm, a great

Fig. 3.—Sanding Commutator Brushes
(Intimate contact between brush and commutator is assured when brushes are carefully sanded in with No. 0 or No. 00 sandpaper. Paper is placed sanded face up and held firmly against the commutator, then pulled circumferentially in the direction of rotation, not across the brush face. Return stroke should be made with the brush lifted from the surface of the sandpaper. Never use emery cloth as metallic particles can short out the commutator bars and puncture the film surface. This brush-holder is of the double brush type.)

improvement can be obtained by adding one or two more brushes per arm (see Fig. 3), because it isn't likely that all the brushes with the same polarity will be off the commutator at the same instant. This does happen, however, in the case of one brush per polarity.

Proper Hardness of Brushes

The question of the proper hardness of brush arises frequently. The softer brushes are more likely to result in better commutation. This is thought to be due to the fact that they wear away more rapidly and present a fresh face to the commutator and therefore maintain a higher contact drop. However, soft brushes have two distinct disadvantages; one is that they wear away more rapidly, and the other is that they are more easily chipped or broken where there is any vibration or pounding present.

It is imperative to use hard brushes on applications where there is any considerable amount of vibration, such as a motor driving a load through spur gears. Gears introduce a high-pitched vibration into the motor armature, which results in chattering of the brushes. Soft brushes will break up under this chattering and are unsatisfactory. The hard brush, however, does not wear away as rapidly as the soft brush and therefore develops a glaze on the face that is not particularly conducive to good commutation.

Double Brush Holder

The invention of the double brush holder has resulted in a marked improvement in commutation (see Fig. 3). This brush holder increases the likelihood of more brushes being in contact with the commutator at any instant and also provides a longer path through which the short-circuited currents of the armature coils undergoing commutation must travel.

The importance of having several brushes per polarity cannot be overemphasized. Motors of 5 hp. or larger should have at least two brushes per polarity. If one brush is knocked off the commutator, the other one is in contact and carries the current. If all the brushes of one polarity are jarred from the commutator there is certain to be an arc formed between the brush and the face of the commutator in order to carry the current.

This problem is particularly true on machines that have but two brush arms, or one brush arm for each polarity. The motor having but two brush arms cannot possibly commutate as well as one having four arms. With two brush arms all of the brushes for one polarity are in contact with the same bars in the commutator. The chances of them being disturbed simultaneously are very great. This should not be taken to mean that it is impossible to operate motors with only two brush arms. The necessary crowding sometimes found in certain types of machinery makes it highly desirable to use but two brush arms. However, the fact still remains that the machine would operate better from the standpoint of commutation if it had four brush arms, or two per polarity.

Replacing Brushes

It is quite easy to replace an old brush with a new one and walk away from it and let it wear itself so that its face has a contour agreeing with that of the commutator. This is particularly true if several brushes are being replaced simultaneously and especially so if they are all being replaced. Sparking with its resultant burning of the commutator is sure to occur if unfitted brushes are applied to the machine. The brushes should be sanded to the contour of the commutator by first grinding the face of the brush with rough sandpaper placed between the commutator and the brush, as shown in Fig. 3. The sandpaper should be pulled in the direction of rotation. At the same time it should be held firmly against the face of the commutator. The return stroke should be made with the brush lifted from the surface of the sandpaper. After the rough grinding has been finished and the brush has the general shape of the surface of the commutator, the remainder of the grinding should be done with fine sandpaper. While it is permissible to apply additional pressure to the top of the brush during the rough grinding only the pressure of the brush finger should be used during the finish sanding.

It is impossible to grind in brushes so they will have a polished face before the machine is operated. However, a simple check can be made to determine whether the brush has been ground in satisfactorily. When it appears that the brush has been ground in properly, the machine should be operated for one or two minutes and then shut down. If upon observation of the face of the brush polished streaks are apparent between the trailing and leading edges, and these streaks form a uniform pattern all the way across the width of the brush face, a proper job has been done and the machine can be operated without fear of trouble so far as this grinding-in operation is concerned.

There are a number of problems that must be faced relative to the operation of motors. One is the riding of the brushes on the commutator. Commutators are never perfectly round. Even when new the manufacturer allows a tolerance of about .001 in. eccentricity. If the machine has operated for a time the eccentricity combined with other irregularities may reach .002 in. or .003 in. Carbon brushes have a resiliency that makes them particularly well suited to follow these irregularities. They must absorb the movement of the commutator face to a degree without moving the entire brush to the same degree. The brush acts as a sort of a spring so that it contracts and expands itself in following the irregularities of the commutator. Some of this motion, of course, reaches the pressure finger and results in a slight motion up and down.

Proper Film on Commutator

The chocolate-brown color so highly desired on the face of the commutator is in reality an ozide film which has been layed down by the brushes. The presence of this film helps to reduce the friction between the commutator and the brushes. However, conditions arise sometimes when this film is irregular and uneven. This results when the brush density is less than about 5 ampere per square inch and may be encountered when a machine operates for a considerable period of time without load. The effect of this irregular film is to make the brushes chatter. Sparking and consequent burning may occur and the brushes may chip and break up. Regardless, operating conditions may make it necessary for a machine to run at light load for considerable periods of time. To insure a satisfactory film under such conditions, select a brush that has a slight abrasive in it. This will permit a proper film to be deposited on the commutator, yet there will be sufficient scouring action to keep it smooth and even.

The presence of uneven film on a commutator can be detected by feeling the

brush as it rides the commutator. If it feels rough, sandpaper the commutator, as this removes the film entirely. If it feels smooth after sanding the evidence is conclusive that it is irregular film that caused the vibration of the brushes. If, however, the sanding operation did not result in a change of vibration of the brushes, then the conclusion can be drawn that such vibration was not due to film, but to irregularity in the surface of the commutator.

Avoid Sanding and Grinding

Frequent sanding and grinding of the commutator should be avoided if at all possible. Sanding a commutator is not too harmful, but grinding with a handstone is particularly injurious because this action actually cuts away small particles of copper. These particles of copper can be, and some always are, drawn into the windings of the armature where they can cause shorts and grounds. This danger is particularly great in machines that have open risers between the bars and the ends of the armature coils. It has been the experience of many users that armatures fail soon after handstoning.

Brush potential curves are a very useful means of determining whether the interpole strength is proper for best results. To obtain values for such a curve it is necessary to use a high resistance voltmeter with a range of about 3 volts. The readings taken are of the voltage drop between the brushes and the commutator. To obtain these one terminal of the voltmeter is attached to the brush rigging and the other to a pointer which must be insulated to permit holding it in the hand. This pointer is gently applied to the rotating commutator, first at a point near the leading edge of the brush where it contacts a bar. A reading is taken. This is point number one on the curve. The pointer is then moved to the center of the brush and a reading taken. This is point number two. A third reading is taken near the trailing edge of the brush and is recorded as point number three. Regardless of the direction of rotation the curve is plotted with point number one to the left and point number three to the right. If the commutating pole strength is too great the curve will slope downward from point one to point three. If the machine is undercompensated then the curve will slope upward from point one to point three. The most desirable curve is one that has a slight downward slope from number one to number three.

Occasionally turns of a commutating coil are removed if the lead happens to be burned off. For example, if the lead is off a coil that has 24 turns, one turn may be unwound in order to provide a new lead. This should not be done for the reason that the removal of one turn in the coil very materially alters its commutating effectiveness. Approximately 80 per cent of the turns on the coil are necessary to suppress the armature mmf before any effect is made by the coil on commutation. Only the last 20 per cent of the turns on the coil are effective in aiding commutation. When the coil has but 24 turns, only about five are effective for correcting commutation. If one turn is removed it changes the effectiveness of the coil 20 per cent. Therefore, it is obvious that this is a questionable practice that will undoubtedly result in sparking and burning of the commutator.

Another source of poor commutation not commonly recognized is inadequate soldering of leads to the commutator bars. This increases the resistance thereby changing the flow of current between the coils and the bars. Thus the bar with the poor joint does not carry its full share of current. If the joint is poor bits of melted solder will be found around these joints.

Another cause of poor commutation is a short in the main field winding. This upsets the equality of the flux around the armature thereby resulting in poor commutation. It is frequently found in motors around the 5-hp. size.

Good commutation insures continuity of service and only results when a planned maintenance program is followed assiduously.

HELPFUL HINTS FOR HANDLING PACKINGS

1. Use packings of correct size. Oversize packings are difficult to install and may cause excessive friction. Undersize packings may be ruined by too much takeup.
2. Where leakage cannot be permitted, provide a means of introducing a sealing or lubricating fluid such as water, oil or grease into the stuffing box.
3. When renewing packings, use all new, clean packing. Worn packings are dry, highly compressed, have high friction, may develop scoring.
4. Remember packing is relatively cheap. Labor and materials necessary to replace worn shafts, sleeves or rods are expensive and critical commodities.
5. Take up bolts evenly, moving to the bolt diametrically opposite, never to the bolt adjacent.
6. Follow up bolts the next day after installing new gaskets. For lines carrying high pressures, the gradual building up of the pressure and continuous follow-up of bolts is recommended.
7. Exercise great care when using gasketing dopes. Never use oil or grease on a rubber or rubber-compounded gasket. Graphite and glycerine or molasses compounds are not injurious to rubber. Never use dope on both sides of gasket—it may cause slippage or blowout.
8. Do not use a heavier sheet packing than is necessary. The thinner the gasketing material, the better.
9. Ring gaskets are preferable to full face gaskets where the flanges are heavy enough to prevent distortion.

Courtesy *Johns-Manville Corp.*

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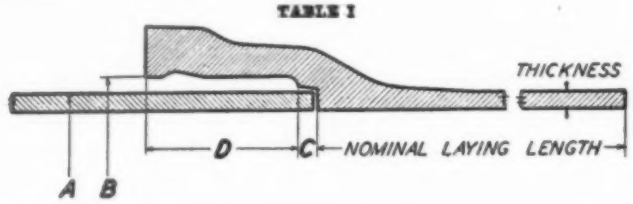
Super-deLavaud Pipe with B&S Joints

Super-deLavaud Pipe with bell and spigot joints is available in all sizes from 3" to 24" and in 12-foot and 18-foot lengths. The joint is particularly adaptable for lead, cement, joint compound or rubber ring assembly. The centering ring insures easy alignment and uniform distribution of the jointing material. The process by which this pipe is produced assures concentricity, evenness of metal and freedom from sand or slag inclusions. The bell and spigot joint, made with lead or other jointing materials, shows ample strength to resist pulling apart. If required to meet service conditions, pipe will be furnished with special linings. All Super-deLavaud Pipe with bell and spigot joints are made to meet Federal Specification WW-P-421. Write for descriptive booklet.

TABLE I



Nominal Diameter Inches	Class of Maximum Working Pressure	Average Thickness	Nominal Laying Length			
			12-Foot Length		18-Foot Length	
			Weight of Pipe Pounds	Weight Per Foot Including Bell	Weight of Pipe Pounds	Weight Per Foot Including Bell
3	150	.33	150	12.7	225	12.4
3	250	.36	165	13.7	240	13.4
4	150	.34	195	16.4	285	15.9
4	250	.38	220	18.4	325	17.9
6	150	.37	315	26.3	460	25.5
6	250	.43	350	29.3	515	28.5
8	150	.42	475	39.4	690	38.3
8	200	.46	510	42.4	745	41.3
8	250	.50	545	45.5	800	44.3
10	150	.47	640	53.3	935	51.8
10	200	.52	700	58.3	1025	56.8
10	250	.57	760	63.3	1115	61.9
12	150	.50	810	67.4	1180	65.6
12	200	.57	905	75.4	1325	73.7
12	250	.62	990	82.5	1450	80.7

Nominal Diameter Inches	Class of Maximum Working Pressure	Average Thickness	Nominal Laying Length			
			12-Foot Length		18-Foot Length	
			Weight of Pipe Pounds	Weight Per Foot Including Bell	Weight of Pipe Pounds	Weight Per Foot Including Bell
14	100	.48	920	76.5	1340	74.3
14	150	.55	1060	88.5	1555	86.3
14	200	.62	1190	99.0	1735	96.3
14	250	.69	1320	110.0	1930	107.3
16	100	.52	1130	94.2	1645	91.4
16	150	.60	1320	110.2	1935	107.4
16	200	.68	1490	124.1	2175	120.7
16	250	.75	1635	136.1	2390	132.7
18	100	.56	1365	113.8	1990	110.6
18	150	.65	1595	132.8	2330	129.6
18	200	.74	1810	150.9	2645	146.9
18	250	.83	2015	168.0	2950	163.9
20	100	.58	1585	132.0	2305	128.0
20	150	.68	1860	155.0	2730	151.0
20	200	.78	2125	177.1	3105	172.4
20	250	.88	2365	197.1	3465	192.4
24	100	.64	2085	173.8	3035	168.6
24	150	.76	2480	206.8	3630	201.6
24	200	.88	2855	237.8	4170	231.6
24	250	1.00	3200	266.8	4690	260.6

Threaded Cast Iron Pipe Centrifugally Cast

Super-deLavaud Cast Iron Pipe is now being manufactured in "iron pipe sizes" with threaded joints to meet the demand for a better and more durable pipe. There are two distinct types of service for which Super-deLavaud Cast Iron Pipe may be used: pressure service for water, gas, steam or similar liquids and gases; and drainage service. Pressure pipe is made to meet the requirements of Federal Specifications WW-P-421 and is supplied in three classes: standard, extra-strong and double-extra-strong. Drainage pipe, used for waste, vent and drain work, is a lighter pipe for use in buildings and elsewhere where a tight screw-coupled joint is needed. Drainage pipe is made to meet the requirements of Federal Specifications WW-P-356.

THREADED CAST IRON PRESSURE PIPE

(Made to meet the requirements of Federal Specifications WW-P-421)

Size Inches	Class	Nominal Length, Feet	Outside Diameter, Inches	Nominal Thickness, Inches	Threads per Inch	Weight per foot Pounds
3	Standard	12	3.50	.36	8	11.08
	X-strong	12	3.50	.45	8	13.45
	XX-strong	12	3.50	.48	8	14.21
4	Standard	18	4.50	.38	8	15.34
	X-strong	18	4.50	.48	8	18.91
	XX-strong	18	4.50	.52	8	20.29
5	Standard	18	5.56	.40	8	20.22
	X-strong	18	5.56	.49	8	24.40
	XX-strong	18	5.56	.53	8	26.15
6	Standard	18	6.63	.43	8	26.13
	X-strong	18	6.63	.51	8	30.59
	XX-strong	18	6.63	.55	8	32.78
8	Standard	18	8.63	.50	8	39.85
	X-strong	18	8.63	.56	8	44.29
	XX-strong	18	8.63	.60	8	47.22

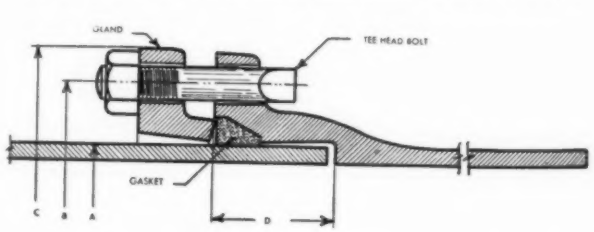
THREADED CAST IRON DRAIN PIPE

For Waste, Vent and Drainage Service

(Made to meet the requirements of Federal Specifications WW-P-356)

Size Inches	Nominal Length Feet	Outside Diameter Inches	Minimum Thickness Inches	Threads per Inch	Weight per Foot Pounds
3	12	3.50	.26	8	9.41
4	18	4.50	.29	8	13.75
5	18	5.56	.33	8	19.05
6	18	6.62	.38	8	26.22
8	18	8.62	.44	8	39.82

U. S. STANDARDIZED MECHANICAL JOINT



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By H. W. CLARK
City Manager, Niagara Falls, N. Y.

IN THE determination of flows in pipe network systems by the Hardy Cross method, it is often desirable, for simplification of the work, to reduce parts of the system to an equivalent size pipe, and use the equivalent value in the computations.

For a quick and easy method of computing equivalent pipes and determining the percentage of flows in pipe loops, the accompanying charts have been prepared.

Method of Preparing Charts

The Williams and Hazen formula[1] was used with the following quantity notations:

The basic formula is $V = CR^{.63} S^{.54}\ 0.001^{-0.04}$
Flow $Q = VA = CR^{.63} S^{.54}\ 1{,}318A.$
where Q = Flow in cubic feet per second (c.f.s.)
C = Williams & Hazen pipe coefficient of roughness.
R = Hydraulic radius of pipe in feet.
S = Slope of pipe = h divided by 1.
A = Area of pipe in square feet = $.7854D^2$
D = Diameter of pipe in feet
h = Head loss in pipe considered
L = Length of pipe considered in feet.

Substituting in the above formula, we obtain—

$$Q = C\ 1.318\ (.25)^{.63}\ .7854\ D^{2.63}\left(\frac{h}{L}\right)^{.54}$$

as the constants are common to all pipes and in equivalent pipes or loops the head loss, h, is the same for each, Q is proportional to $\frac{D^{2.63}}{L^{0.54}}$ for pipes of equal roughness.

This is the same method given by James A. Conklin in an article in *Engineering News-Record*[2] in which he designated the quantity $\frac{D^{2.63}}{L^{0.54}}$ as the Pipe Factor, although he used the Chezy formula with the factor equal to $\frac{D^{2.5}}{L^{0.5}}$.

In order to construct the charts the $D^{2.63}$ values for nominal diameters of cast iron pipe were computed and spotted on the Y axis of logarithmic paper, then through each of these points were drawn the parallel lines $1/L^{0.54}$ using 1,000 foot units for L. At the left of the diagram is read the factor for any corresponding pipe diameter and length.

For convenience in use two charts on 8½x11 inch logarithmic paper have been prepared rather than preparing a large unwieldy chart. Obviously the charts overlap and the choice of chart in the overlapping sections depends on the length of pipe involved in the computation.

Demonstration of Chart Use

In demonstrating the use of the diagrams, some published examples will be used, as most readers will probably be familiar with the examples.

Example A

In D. R. Taylor's excellent article on the Hardy Cross method, in *Water Works and Sewerage*,[3] he uses the following circuit:

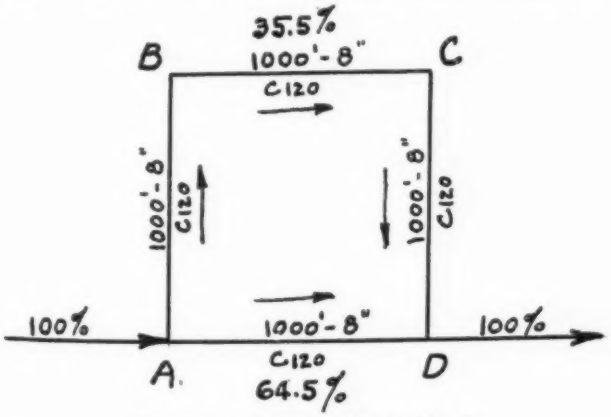

Fig. 1—Example A—Simple Circuit

The direction of flow is indicated by the arrows. From Chart 1 Factor (F) is determined as follows: Look for the 1000 foot line at bottom of diagram, follow this line vertically to diagonal marked 8 in., from this intersection point follow horizontally to the left scale and read .345. In the same manner follow the 3000 foot line vertically to the 8 in., diagonal, thence left from that point and read .19.

Add these two quantities and divide the sum into each, obtaining the percentages shown. These percentages check those of the original problem, after its two calculations.

The sum of these factors, 0.535, intersects the 8 in. diagonal line at 445 feet.

Factor F = 0.535 = 445 ft. in 8 in. pipe, the equivalent pipe.

AD = 1000 ft. of 8 in., F = .345	64.5%
ABCD = 3000 ft. of 8 in., F = .19	35.5%
.535	100%

Inasmuch as the Q's are expressed in percentages where the head losses are desired, Q may be used as g.p.m., c.f.s. or m.g.d.

We can now check the above problem using Hardy Cross factors from Taylor's article,[3] Fig. 2.

The k for 1000 ft. of 8 in. with (C 120) = 6.0; for 3000 ft. of 8 in. with (C 120) = 18.0.

The formula for head loss by Doland's formula[4] is

$$h = \frac{k}{100{,}000}\ (\text{gpm.})^{1.88}$$

This formula can be simplified by taking $100^{1.88}$ out of the parenthesis and dividing by 100,000; it then can be written

$$h = k\ (.05012)\ (.01\ \text{gpm.})^{1.88}$$

Calling the total Q, 1000 gpm. the flow in AD will be 645 gpm., in ABCD 355 gpm. and in the equivalent pipe 1000 gpm.; then the head loss in the paths of the loop and in the equivalent pipe is found as follows:

Path AD	h = 6 (.05012)	$(6.45)^{1.88}$	= 9.45
Path ABCD	h = 18 (.05012)	$(3.55)^{1.88}$	= 9.43
Equiv. pipe 445 ft. of 8 in.,	k = 6 (.445)		= 2.67
	h = 2.67 (.05012)	$(10.)^{1.88}$	= 9.44

This agreement of the head loss "h" appears close enough for any practical purpose.

Example B

Three parallel pipes are just as simple to figure as shown below, using the same steps as in Example A

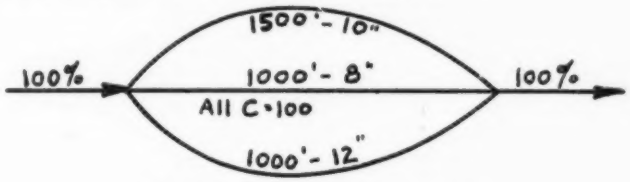

Fig. 2—Example B—Parallel Pipes

1500 ft. of 10 in. F = .50	=27.1%
1000 ft. of 8 in. F = .345	=18.7%
1000 ft. of 12 in. F = 1.000	=54.2%
1.845	100%

F 1.845 = 135 ft. of 10 in.; 325 ft. of 12 in.; or 1300 ft. of 16 in.

These values may be checked by the same method given for finding h in Example A.

Example C

The chart values check closely the values given in tables computed by W. E. Howland and F. Farr, Jr.[5] shown by the following example.

Take 500 ft. of 8 in. pipe F = .5 from Chart I.

Reading across the .5 line where it intersects the 10 in., 12 in., and 16 in. pipe lines we find the equivalent lengths to be

10 in. = 1,480 ft.
12 in. = 3,600 ft.
16 in. = 14,600 ft.

From the Howland & Farr table, page 238[5], mentioned above, we find the equivalent values of 1 ft. of 8-in. pipe in the other sizes to be

1 ft. of 8 in. = 2.965 ft. of 10 in.
1 ft. of 8 in. = 7.205 ft. of 12 in.
1 ft. of 8 in. = 29.25 ft. of 16 in.

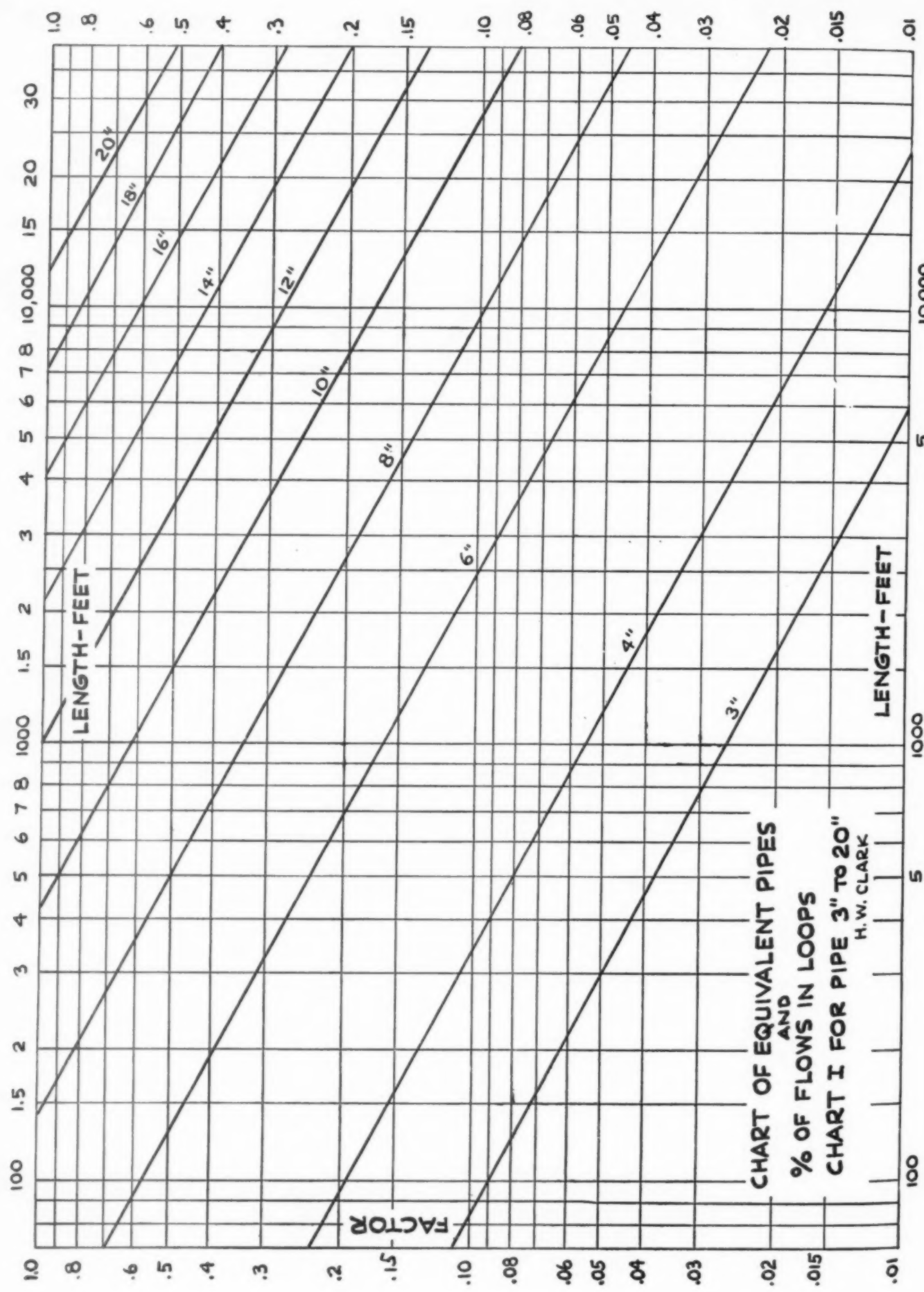

Chart I—Equivalent Pipes and Per Cent Flows in Loops for Pipes 3-in. to 20-in.

Chart I—Equivalent Pipes and Per Cent Flows in Loops for Pipes 3-in. to 20-in.

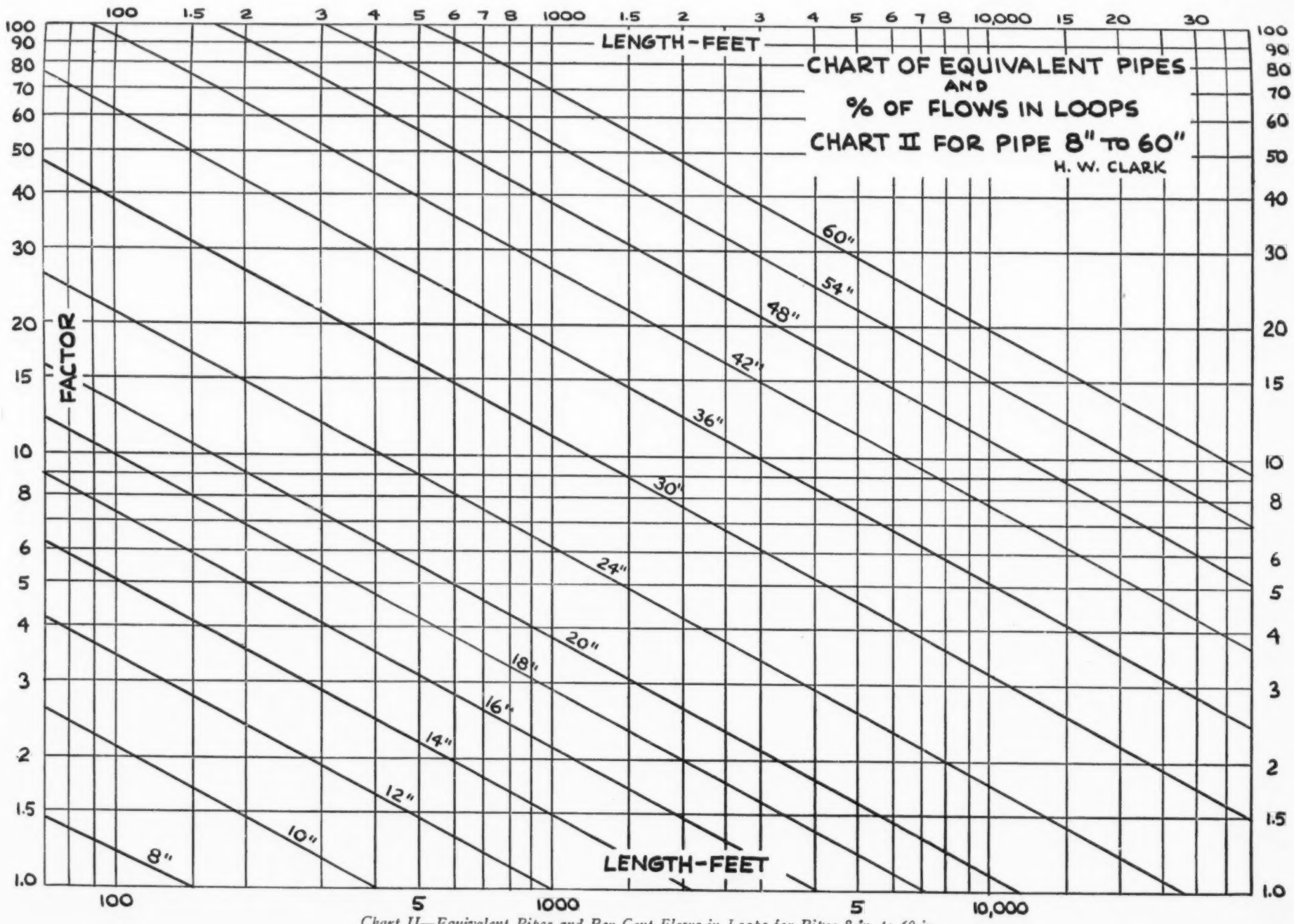

Chart II—Equivalent Pipes and Per Cent Flows in Loops for Pipes 8-in. to 60-in.

Multiplying each of these values by 500 we obtain, respectively, 1482.5 ft. of 10 in., 3602.5 ft. of 12 in. and 14625 ft. of 16 in., which, considering the size of the chart readings, is a pretty fair check.

Example D

Pipe circuits with crossover pipes and intermediate take outs are not subject to exact analysis, but many of them can be closely approximated and many steps saved, as the following problem will show. It is not claimed, however, that all circuits will work out as well, for with complicated systems good judgment and experience in the method of computation will be the best guide.

Let us take D. R. Taylor's two-circuit problem from the same article[3] previously mentioned.

Using the same steps as outlined in Example A the following data are obtained:

AB = 3000 ft. of 12 in.	F = .55	56.7%
AFES = 5000 ft. of 12 in.	F = .42	43.3%
	.97	100.0%

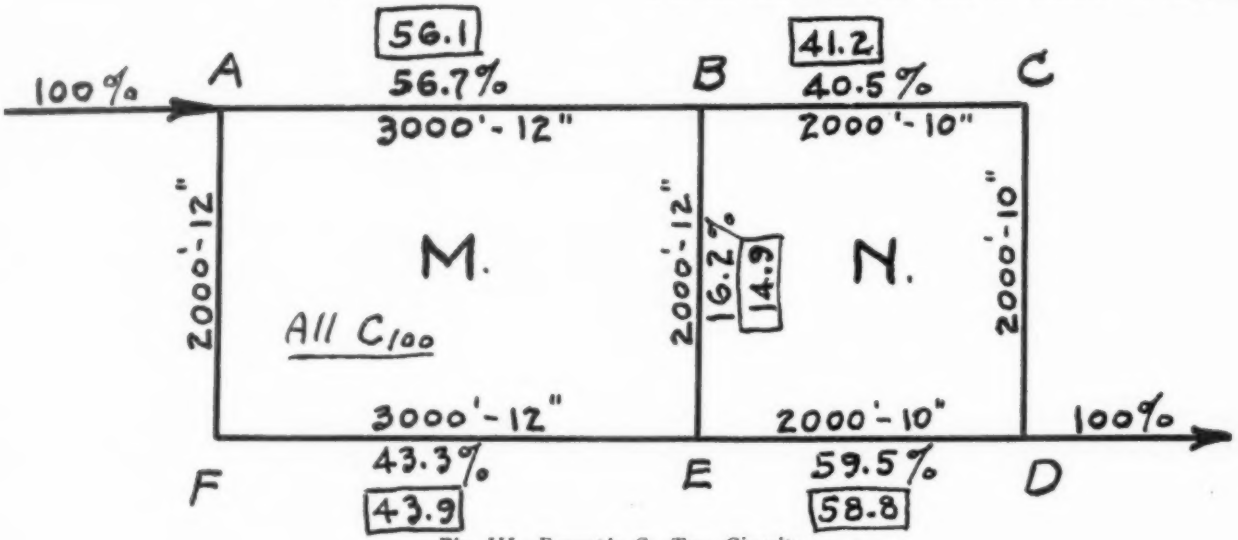

Fig. III—Example C—Two Circuits

BCD = 4000 ft. of 10 in.	F = .29	40.5%
ED = 2000 ft. of 10 in.	F = .425	59.5%
	.715	100.0%

The crossover pipe is not considered in the first approximation, only the outside circuit.

Place the percentage flows as computed on the diagram and we find BE to be 16.2 per cent.

Computing the circuits by the Hardy Cross method, a correction in M of —.6 per cent clockwise is found, and in N of +.7 per cent clockwise on the first computation. These are within .1 per cent of the final value.

The pipe BE has two corrections, both being minus, with the final value of 16.2—.6—.7 = 14.9.

Example E Where C Varies in the Loops

In the foregoing examples the C of all pipes has been considered equal but pipes of different C values can also be compared by converting both to the same C and proceeding as demonstrated.

Consider the first problem again, using C = 100 for AD and C = 120 for ABCD.

The change can be either way, the method to pursue as follows:

Change AD (C = 100) to C = 120 by multiplying the length 1000 ft. by $\left(\frac{120}{100}\right)^{1.85} = 1000 \times 1.4 = 1400$ ft.

If the change had been to C = 100 for the C = 120 pipe, ABCD = 3000 ft. (C = 120) would have been divided by 1.4 and the resulting length used with the 1000 ft. for AD and C = 100 for both.

We will use the C = 120 values, then

AD 1000 ft.	(C = 100) = 1400 ft.	(C = 120);	F = .288	60.2%
ABCD 3000 ft.	(C = 120) = 3000 ft.	(C = 120);	F = .190	39.8%
			.478	100.0%

Equiv. pipe F (.478) = 550 ft. 8 in. (C = 120)

Check computations.

AD k for 1400 ft. of 8 in. (C = 120) = 6 (1.4) = 8.4
ABCD k for 3000 ft. of 8 in. (C = 120) = 6 (3.0) = 18.0
Equiv. pipe k for 550 ft. of 8 in. (C = 120) = 6 (.55) = 3.3

Assuming 1000 gpm total flow.

AD h = 8.4 (.05012) $6.02^{1.85} = 11.61$
ABCD h = 18. (.05012) $3.98^{1.85} = 11.60$
Equiv. Pipe h = 3.3 (.05012) $10^{1.85} = 11.68$

It will also be noted that the k value for 1400 ft. of 8 in. (C = 120) is 8.4 which is the same value for 1000 ft. of 8 in. (C = 100) 8.4 × (1.0) from Fig. 2[3] values of k.

For changing from C = 100 to any other value of C, multiply by the following factor; if changing from C = 120, etc., to C = 100, divide C = 120, etc., by the factor; these factors are simply $\left(\frac{Cx}{C100}\right)^{1.85}$

Where C =	90	100	110	120	130	140
f =	.823	1.0	1.193	1.40	1.625	1.863

Conclusions

The author has used the foregoing method in computing circuits and found it helped greatly in reducing the computations required, in some instances even giving initial results within a few per cent of the final.

One is cautioned, however, that complicated circuits and those with take-offs must be examined carefully, as small increment corrections will sometimes throw them out of balance instead of toward convergence. Large pipes with low resistance factors in loops with smaller pipes of high resistance are very tricky in adjusting flows.

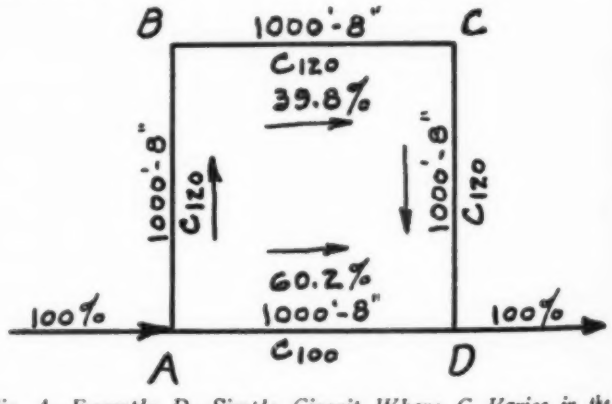

Fig. 4—Example D—Simple Circuit Where C Varies in the Loop

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1. Williams & Hazen—Hydraulic Tables, John Wiley, New York (1933 Edition).

2. James A. Conklin—Flow in Pipe Networks by Direct Determination—*Eng. News-Rec.* 128-41 (1942).

3. D. R. Taylor—The Hardy Cross Method—*Water Works & Sewerage* (1943).

4. J. J. Doland—Simplified Analysis of Flow in Water Distribution Systems—*Eng. News-Record*, 117-475 (1936).

5. W. E. Howland & F. Farr, Jr.—Computation of Equivalent Lengths of Parallel Pipe Systems—*Jour. AWWA*, 33-237 (1941).

6. Hardy Cross—Analysis of Flow in Networks of Conduits or Conductors—*Univ. of Illinois Bul.* No. 268 (1936).

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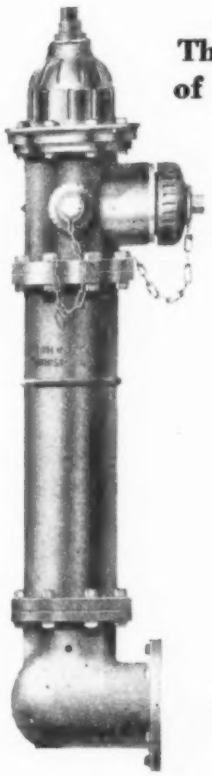

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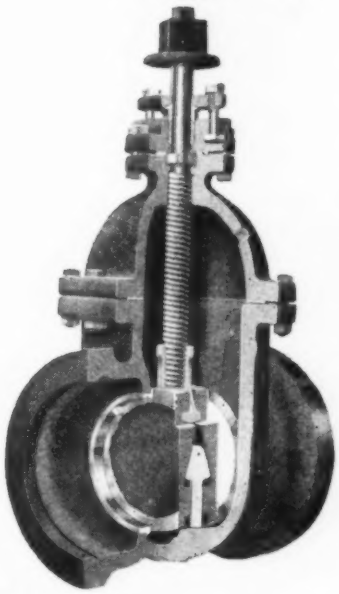

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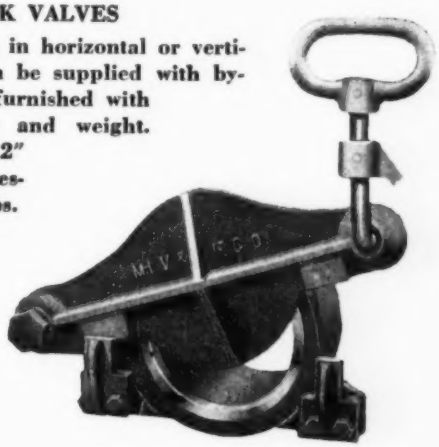
Class	Working Pressure	Hydrostatic Test
AA	0 - 25 lb.	50 lb.
A	26 - 50 lb.	100 lb.
B	51 - 100 lb.	200 lb.
C	101 - 200 lb.	350 lb.

CHECK VALVES

For installation in horizontal or vertical position. Can be supplied with by-passes. May be furnished with an outside lever and weight. Sizes from 2″ to 12″ for working pressure up to 200 lbs.



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Johns-Manville answers 13 QUESTIONS frequently asked about TRANSITE* PRESSURE PIPE

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1. What is Transite Pipe's flow coefficient?

The conservative "C" value for Transite Pressure Pipe is 140, based on the Williams and Hazen formula.

2. Is Transite Pipe subject to tuberculation?

No! Transite's asbestos-cement composition is assurance that its original high carrying capacity cannot be reduced by tuberculation.

3. What economic advantages result from Transite's immunity to tuberculation?

Transite's complete freedom from tuberculation provides substantial savings in the initial cost of the system as well as in maintenance and operating costs.

It means, first, that you don't have to resort to larger-than-necessary pipe to offset the progressive reduction of carrying capacity caused by tuberculation. Without "over-designing," you can specify minimum size pipe to do the job intended—providing flow for present needs and those incident to increasing population and industrial growth.

It means also that the pipe you install *now* won't have to be cleaned, lined, reinforced with additional mains, or replaced altogether because tuberculation has clogged its interior and reduced its original carrying capacity.

4. How about pumping costs?

Here again, Transite's non-tuberculating asbestos-cement structure pays off. In a Transite line it is not necessary to increase pumping pressures to compensate for the effects of tuberculation. Thus, pumps can operate at higher efficiencies—an economy which is helping to keep down water rates for the taxpayer in many communities served by Transite water lines.

5. Does Transite withstand corrosive soils?

In just about every type of soil, Transite has demonstrated its superior resistance to soil corrosion. This statement has been substantiated by impartial comparative studies conducted both by government and private agencies.

For example, one series of studies revealed that the corrosive effect of many types of aggressive soils on Transite Pipe was practically negligible after exposure over an extended period. Other pipe materials subjected to the same tests were substantially affected by corrosion, but the strength of the Transite Pipe actually increased under identical test conditions.

Another indication of Transite's unusual corrosion resistance is its performance in coal mine service. Here, the conveying of acid mine drainage waters is a severe test of the corrosion resistance of any pipe—so severe that the life of a drainage line is sometimes measured in terms of months or only weeks. Yet under these conditions some of the earliest coal mine installations of Transite Pipe, laid more than 12 years ago, are still in use today. Several of them have

outlived the mines in which they were originally installed and are now serving in new operations.

6. What about electrolysis?

Electrolysis is no problem in a Transite water line, for Transite, being non-metallic, is immune to electrolytic action. You can lay this pipe alongside street car or power lines or in the proximity of power stations, with complete confidence that it cannot be harmed by stray ground currents.

7. How about strength?

Three factors contribute to the high uniform strength of Transite Pipe:

1. Its tough, strong asbestos-cement composition (asbestos fibers have a tensile strength as high as 400,000 lbs. per square inch).
2. The method used to form the pipe during manufacture—it is built up under tremendous pressure into a dense, homogeneous structure.
3. The special curing process which follows the forming operation and develops maximum strength in the finished product.

And, Transite's resistance to virtually all types of soil corrosion is assurance that the strength which is built into this pipe at the factory *is built in to stay!* Its record of successful performance under adverse conditions is proof that Transite *maintains* its strength.

8. Does Transite Pressure Pipe provide tight joints?

Yes. Due to the Simplex Coupling, joint troubles in Transite lines are few and far between.

The design of the Simplex Coupling is simplicity itself. Consisting only of a Transite sleeve and two rubber rings, it is, in effect, a factory-made joint which is merely *assembled* on the job. The operation requires only that the sleeve be pulled over the ends of the pipe, which rolls and compresses the rubber rings into place between sleeve and pipe. Thus rubber, which provides one of the most effective of seals, safeguards the tightness of the line. Furthermore, each joint is readily checked for proper assembly as the pipe is laid, providing advance assurance that the line will pass leakage tests.

And because it is a *flexible* joint, the Simplex Coupling offers an important additional advantage after the pipe is in service. Its flexibility helps compensate for soil movement and other stresses to which the pipe may be subjected, thereby helping protect against joint leakage. This also provides a safeguard against washing away of the supporting soil and possible pipe breakage.

9. How much deflection is possible at the joints?

A deflection of 5° or more at each joint is possible on most installations of Transite Pressure Pipe—an important advantage when laying pipe around curves and in hilly terrain.

10. What installation economies does Transite provide?

It is an exceptionally economical pipe to handle and install. The lightweight sections make the use of mechanical handling equipment unnecessary except for the largest sizes. The Simplex Coupling employed assures a fast assembly job in the trench. Under most conditions, the pipe can be laid as fast as the trenching machine can do its work. In fact, because of this feature, many installations have been made with the same foreman supervising both excavation and pipe laying. And because of its rapid assembly, pipe installation jobs are completed sooner, minimizing street tie-ups and traffic annoyances.

11. Are service connections easily made?

Yes. Transite Pipe can be drilled and tapped with standard tapping tools and methods. Connections are strong and watertight. Connections to valves and fittings, cut-ins for valve insertions and branch take-offs, etc., are made by conventional methods.

12. In what diameters is Transite Pressure Pipe made?

Designed for both supply and distribution lines, Transite is made in diameters ranging from 3" up to 36".

13. What working pressures will Transite handle?

Four classes are made—for working pressures of 50, 100, 150 and 200 pounds per square inch. To assure an ample factor of safety, each length of pipe and each coupling is factory tested before shipment at four times its specified working pressure.

Have you any further questions?

If you would like additional information about Transite Pressure Pipe, send for brochure TR-11A. Or if you prefer, ask to have a Johns-Manville representative call. Address Johns-Manville, Box 290, New York 16, N. Y.

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VENTURI METER

The Venturi Meter is recognized as the standard for measuring the flow of water, sewage and air in main pipe lines. It consists of a Venturi Tube installed in the pipe line and a Type M Register, Indicator, Recorder or other instruments connected to the Tube by small pressure pipes. The design of the Herschel Standard Venturi Tube produces a maximum recovery of pressure, thus having the economic advantage of lowering pumping costs. Bulletin D10-110.

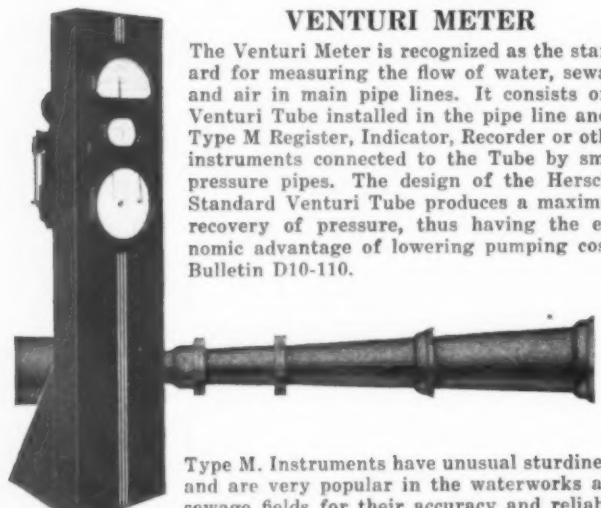

Fig. 1. Herschel Standard Venturi Tube and Type M Register - Indicator-Recorder

Type M. Instruments have unusual sturdiness, and are very popular in the waterworks and sewage fields for their accuracy and reliability. Important features are a 10" indicator at eye level with uniformly spaced scale; improved integrator; 12" uniformly spaced charts; white pyralin dials and interior illumination. This instrument furnishes three kinds of information: continuous indication of the flow; a permanent record on the 12" chart; and the total quantity on a 3½" counter dial. All dials graduated in direct reading units for the particular line being metered. Bulletin D7-200.

PROPELOFLO METER

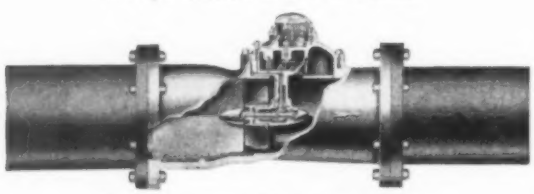

Fig. 2. Propeloflo Meter flange mounted. Available also in bell and spigot model.

This improved, propeller-type flow meter for main water line metering introduces many new features including streamline Venturi design. A self-contained unit easily installed; registers total flow directly in gallons. Accurate throughout wide range; low loss of head; low first cost and upkeep. Bulletin 350A.

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A small, medium-priced mechanical instrument which measures the flow of liquids, steam, air and gas through standard differential producers such as Venturi Tubes, Nozzles and Orifice Plates. Also furnished with Kennison Open Flow Nozzles, Parshall Flumes and Weirs or for open channel flow measurement. This sturdy, accurate and dependable instrument is available in a variety of mountings and dial combinations and for remote operation of secondary instruments or the automatic proportional control of dry or liquid chemical feeders. Three separate easy to read dials are compactly arranged in the space usually required for only two dials:

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(2) Uniformly spaced 10-inch segmental Indicator.
(3) Uniformly graduated 12-inch Chart Recorder.

Bulletin D11-300.

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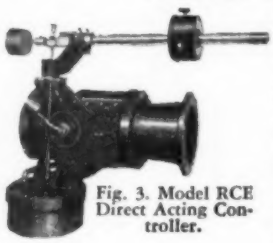


Fig. 3. Model RCE Direct Acting Controller.

Several models are offered, each designed for particular conditions, to automatically maintain constant flow rate through the filter bed. Model RCE features smooth, unwavering control; low loss of head; sturdy, balanced valve; compact flexible design; powerful "floating piston" and exclusive molded tapered piston seal. All rotating bearings anti-friction. Modified for higher pressures and velocities, these controllers serve as Wash Water Controllers or in the effluent lines from pressure filters. Bulletin 321A. Models RCA and RCB are hydraulically operated. Model RCA has a gate type valve with specially shaped seat. Model RCB has a special double butterfly valve of rectangular section and is used for lines over 12" diameter. Bulletin 325A.

FILTER GAUGES

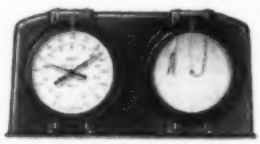


Fig. 4. Loss of Head and Rate of Flow Gauge.

Filter Gauges are usually mounted on the operating table convenient to valve controls, but can be mounted on self-supporting floor stands where desirable. The indicator and chart display accurate information gathered by the dependable Builders Diaphragm Pendulum Units. Attractively designed for modern plant interiors. Available in many combinations to indicate and/or record Rate of Flow, Loss of Head, Liquid Level and Sand Expansion. Special types are custom built to engineers' specifications. Bulletin D1-450.

METERS FOR OPEN FLOW

Flow through partially filled pipes, sewers and open channels, using a Kennison Nozzle, Parshall Flume, or Weir, where there is a wide range of flow rates, can be accurately measured using standard Builders instruments. The Kennison Nozzle is particularly successful in measurement of raw sewage sludge and trade waste. It is available in sizes from 6" to 36". Bulletins D5-135 (Kennison Nozzles) and D2-100 (Parshall Flume).

CHRONOFLO TELEMETERS

Fig. 5. Chronoflo Receiver.

Chronoflo Telemeters bring to a central operating point, from widely scattered locations, accurate records of flow, level, pressure, temperature, gate position and weight. Chronoflo Transmitters are electrically connected by a simple two-wire circuit to Chronoflo Receiving Instruments. Automatic proportional control of Chemical Feeders in step with rate of flow is accomplished by a simple attachment to a standard Chronoflo Transmitter. Bulletin D10-230.

SELECTION OF MAIN LINE METERS

With a Set of Charts Designed to Simplify the Problem

By J. C. THORESEN, *Engineer*
Builders-Providence, Inc., Providence, R. I.

IN the selection of suitable meters for main gravity and pump discharge pipe lines, price is too frequently the sole determining factor, whereas a closer examination of the proposals may demonstrate that such a decision would be definitely improper from an economic standpoint. This does not assert that the lowest priced equipment is never warranted, but simply that experience has repeatedly emphasized the importance of making an evaluation of design and specifications. These include the reputation of the meter; its anticipated life; materials, workmanship and finish; probable cost of maintenance; character of information furnished, such as size of dials and charts, uniformity of graduations, whether direct reading in units of flow or indirect in nominal values or differentials; and, most important, the friction loss induced. It is not the purpose of this paper to comprehend all of these, but to devote the discussion to the last mentioned, the friction loss, resulting in loss of pressure, or power, or both, occasioned by the insertion of the flow measuring element in the pipe line.

For gravity lines under low heads, the friction loss of the proposed meter should be carefully studied to determine whether objectionable in its effect on the hydraulic gradient. In a pump discharge line, friction loss alone may not be of significance, but when considered together with pipe line size and hours of operation, the combination of these three factors may be of great monetary importance in a 12 months' period.

In general, main line meters operate on the inferential principle: that is, the flow rate is "inferred" from the difference in pressure induced by a "restriction" placed in the pipe line and transmitted to the recording instrument, through which no flow occurs. It is readily appreciated by all water works men that elbows, tees, partly closed valves or other fittings add to pipe line friction loss and must be reckoned with in the design of the system. So it is with meters. Whether water is first lifted to a reservoir or pumped directly through the pipe line, the additional load is reflected back upon the pumps, requiring provision for more horsepower than would otherwise be required, and power always costs money.

Forms of Differential Producers

There are only a few forms of "differential producers" for insertion in pipe lines. Of these, the Venturi Tube and Orifice Plate are by far the most widely employed. They are entirely unlike in their construction.

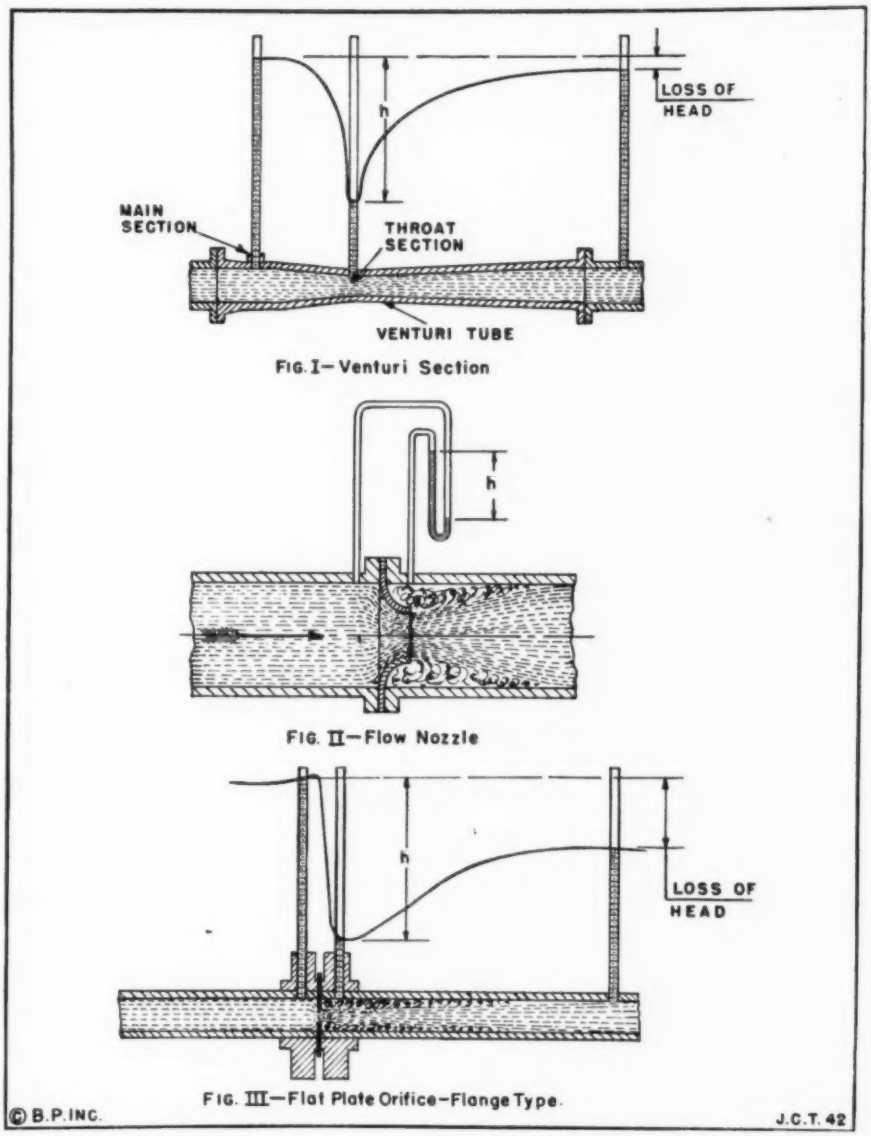
The Orifice (Fig. III) is essentially a flat plate between flanges of the main pipe. As the flowing water approaches the Plate, the stream lines undergo an abrupt (nearly 90 degree) change in direction; then, after passing through the Orifice opening, start to expand again until finally the pipe line is completely filled. As would be expected, there is considerable turbulence in the surrounding annular space on both sides of the Orifice occasioning a relatively high over all pressure head loss.

The Venturi Tube

The Herschel Standard Venturi Tube (Fig. I) is of "streamlined" proportions. It forms a section of the pipe line. As the flow progresses from the inlet section there is a rapid acceleration of velocity with decreasing pressure until the "throat" is reached, where the speed of flow may be ten or more times as great as at the inlet. After passing the throat, the velocity is decelerated through a gently expanding outlet cone, with consequent gradual regain in pressure. The velocity distribution is equal across the throat and the rate of flow is based upon the throat area. Throughout this interchange of velocity and pressure, there is wall guidance of the flowing stream and turbulence is avoided. For this reason the standard Venturi Tube has definite advantage over the Orifice in the important matter of pressure loss.

Orifice Plates and Nozzles

Fig. II illustrates one form of the "Nozzle", of which there are various designs. The type shown is slightly better than the Orifice in pressure loss characteristics, but not enough to warrant separate consideration. The graph (Fig. IV) shows the comparison of friction loss for the three differential producers mentioned. The steep slope of the two upper curves for the Orifice and Nozzle are in sharp contrast to the Venturi curve: in the Venturi the friction loss changes but a few per cent for a wide range of the throat diameter to inlet diameter ratio (d_2/d_1) in contrast with the other two devices. It will be observed that



Figs. I, II and III—Three Forms of Differential Producers for Main Line Metering.

the friction loss is in per cent of the Orifice or Venturi differential head (i.e.—head at inlet minus head at throat) which may reach a maximum of 10 to 20 feet of water, if not more, for accurate metering.

A reduction in the unrecoverable loss of head through an Orifice or Nozzle to a value comparable to the Venturi Tube, by increasing the size of the orifice or nozzle opening, also reduces the differential pressure at any given rate of flow and results in a decreased measuring range.

Example: 10-in. Herschel Standard Venturi Tube with 4-in. throat. Rate of flow 1,400 gallons per minute. $d_2/d_1 = 0.4$.

By following directions for Example 3 given on the Rate of Flow Alignment Diagram, Fig. V, we find differential head = 240 inches of water.

Loss of Head, Fig. IV, 11½% of 240 inches = 27.6 inches of water.

Assume that the average meter recording instrument will not respond accurately to changes in differential of less than .010 inches of water.

Minimum differential equals 1.00 inches for performance accuracy of 1% of differential.

Rate of flow varies as square root of differential.

$\therefore$ accurate measuring range of Venturi $= \sqrt{\frac{240}{1}} =$

15.5 to 1 or 1,400 to 90 G.P.M.

For same unrecoverable loss of head of 27.6 inches Orifice Plate opening would be 7.06 inches (approx.) or $d_2/d_1 = .7$.

Differential pressure for 1,400 G.P.M. = 53.4 inches of water.

Loss of Head (Fig. IV) = 51¾% of 53.4 inches of water = 27.6.

Accurate measuring range of Orifice Plate $= \sqrt{\frac{53.4}{1}} =$

7.3 to 1, about one-half that of Venturi.

Having now established briefly the characteristics of two principal differential producers, let us determine their practical significance by recourse to common problems met in water works practice.

Horsepower is measured in foot-pounds of work performed in a definite time period. If we raise a 50 pound weight a distance of one foot, the work done is 50 foot-pounds. By analogy, if we have 8,340 pounds of water (approximately 1,000 gals.) per minute flowing through a pipe line and one foot of head is lost by pipe friction, the result is the same as though the water had been raised one foot, or energy is being expended at a rate of 8,340 foot-pounds per minute.

Practical Problems

Two practical problems are cited to illustrate the method of determining the comparative cost of using Venturi Tubes and Orifice Plates.

Problem A

Water pumped through line containing 16 inch Herschel Standard Venturi Tube with 6½ inch throat.

Average rate, 3,700 G.P.M.

Electric-driven pump with over-all (wire to water) efficiency of 70%.

Average operating period 15 hours per day for 360 days per year.

Power cost 1½c per K.W.H.

$$d_2/d_1 = \frac{6.5}{16.0} = .405$$

Using Alignment Chart, Fig. V, Venturi differential = 240 inches = 20 feet of water.

From Fig. IV, corresponding friction loss = 11½% of 20 feet = 2.3 feet.

3,700 G.P.M. × 8.34 = 30,858 pounds.

Energy Loss = 30,858 × 2.3 = 70,973 foot-pounds per minute; and $\frac{70,973}{33,000} = 2.15$ horsepower.

Equivalent electrical loss = 2.15 × .746 = 1.605 kilowatts.

Corrected for motor and pump efficiency $\frac{1.605}{.70} = 2.29$ KW.

Cost of power per year

= Hours × KW. Loss × Cost per K.W.H. = $185.00
(360 × 15) × (2.29) × (.015) = 1

From Fig. V, an Orifice Plate for same maximum flow would have an opening 8.25 inches diameter.

$$d_2/d_1 = \frac{8.25}{16} = .515.$$

Loss through Orifice = 71% of differential = .71 × 20 = 14.20 feet.

$$\text{HP. Loss} = \frac{14.20}{2.3} \times 2.15 = 13.3.$$

$$\text{Cost of Power per year} = \frac{13.3}{2.15} \times \$185.00 = \$1,145.00.$$

Cost of 16 inch Venturi Tube	$640.00
Cost of 16 inch Orifice Plate and Flanges	65.00
Difference	$575.00

Saving in power consumption by using the Venturi Tube, $1,145.00 — $185.00 = $96.00 per year.

Omitting depreciation, $\frac{960}{575} = 167\%$ return on investment.

Conclusion: In comparison with the Orifice, the Venturi Tube would pay for itself in about six months and should be selected.

Problem B

Motor-driven pump operating 15 hours per day, 350 days per year = 5,250 hours per year.

Discharge line, 4.00 inches.

Wire to water efficiency, 60%.

Cost of power, 2c per K.W.H.

Pump discharge average rate, 100 G.P.M.

4.00 inch Venturi Meter designed for a maximum of 350 G.P.M.; throat diameter, 2.00 inch.

$$d_2/d_1 = \frac{2.00}{4.00} = .50.$$

Following the procedure outlined in "Problem A" and using Alignment Chart, Fig. V, differential pressure across the Venturi connections for 100 G.P.M. is 18¾ inches water.

From Fig. IV, friction loss = 10½% of 18¾ inches = 1.97 inches or .164 feet.

$$\text{H.P. loss} = \frac{100 \times 8.34 \times .164}{33,000} = .00415.$$

Substituting an Orifice Plate for the Venturi Tube the 18¾ inches differential (Fig. V) gives Orifice opening of 2.49 inches

or $d_2/d_1 = \frac{2.49}{4.00} = .622.$

From Fig. IV, friction loss = 65.0% of 18¾ inches = 11.32 inches or .945 feet.

$$\text{H.P. loss} = \frac{.945}{.164} \times .00415 = .024.$$

.024 — .00415 = .0199 excess of H.P. loss of Orifice over Venturi.

.0199 × .746 = .0148 K.W.

Corrected for pump efficiency.

$$\frac{.0148}{60} = .0247 \text{ K.W.}$$

Difference in cost of power per year of operation = cost per K.W.H. × electrical loss × hours of operation = .0199 × .0247 × 5,250 = $2.58.

Cost of 4 inch × 2 inch Venturi Tube	$125.00
Cost of Orifice Plate and Flanges	30.00
Difference	$ 95.00

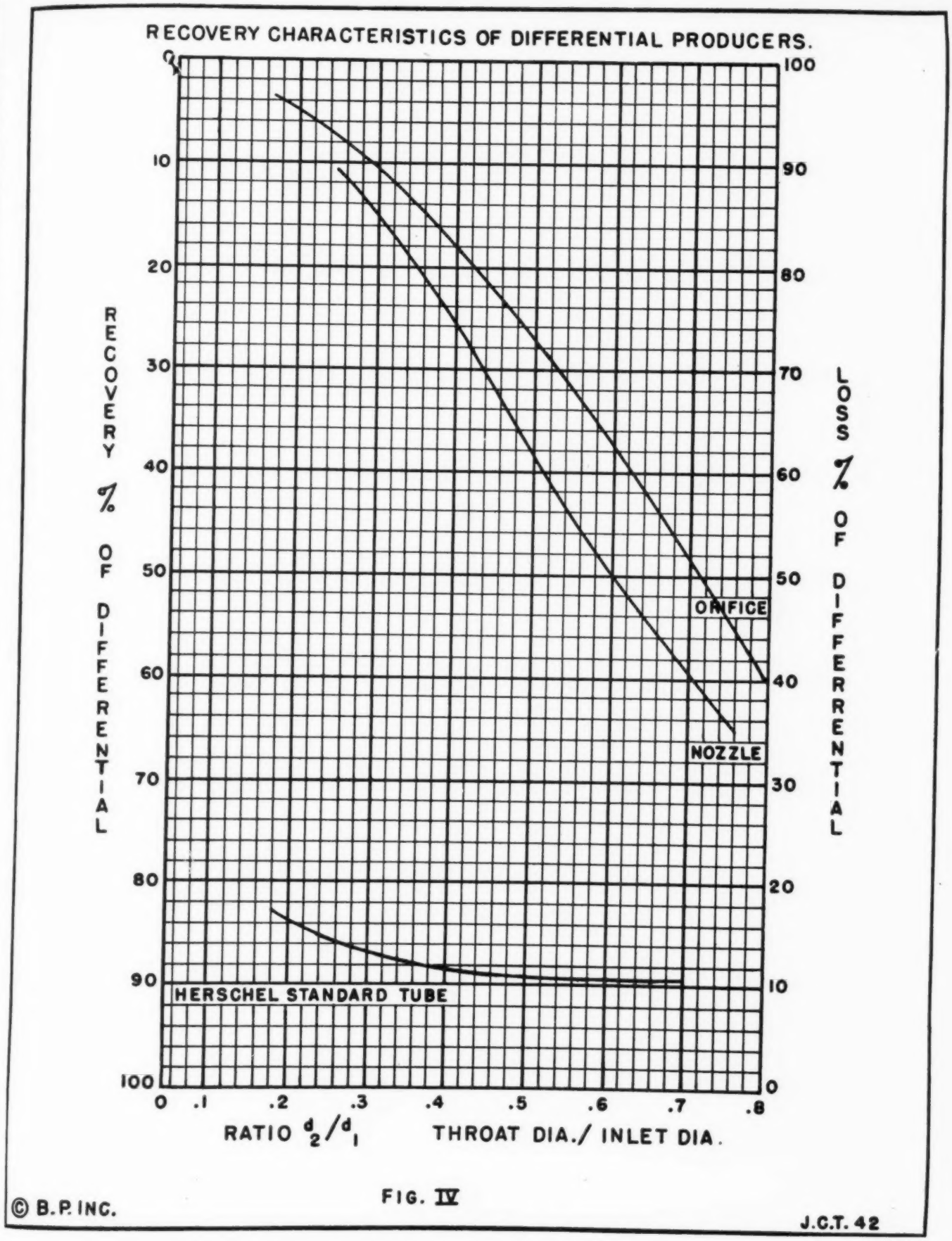


Fig. IV—Recovery Characteristics of the Venturi Tube, Nozzle and Orifice.

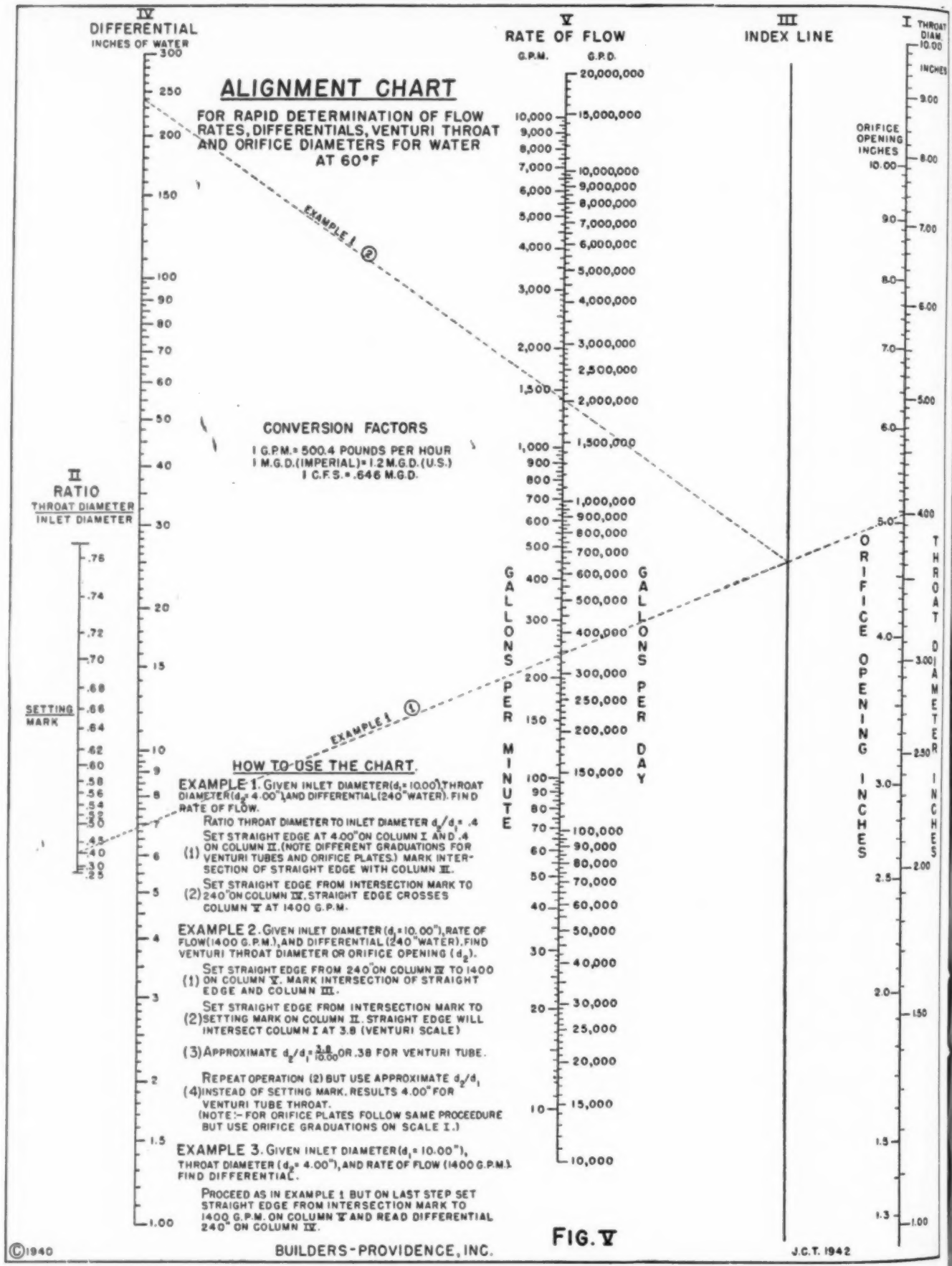


Fig. V—Chart for Rapid Determination of Flow Rates, Differentials, Venturi Throat and Orifice Diameters.

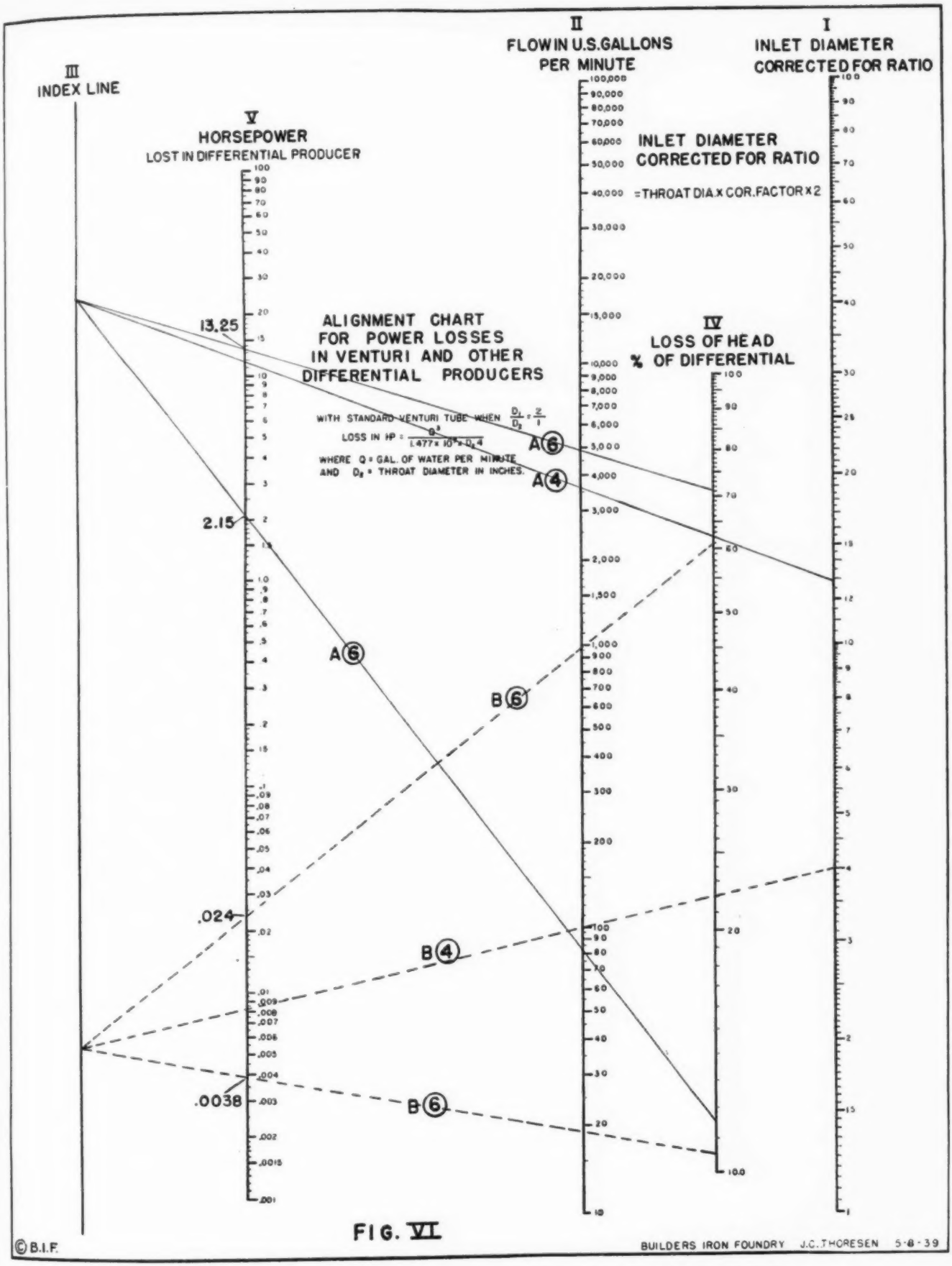


Fig. VI—Chart for Power Losses Through Venturi Tubes and Other Differential Producers.

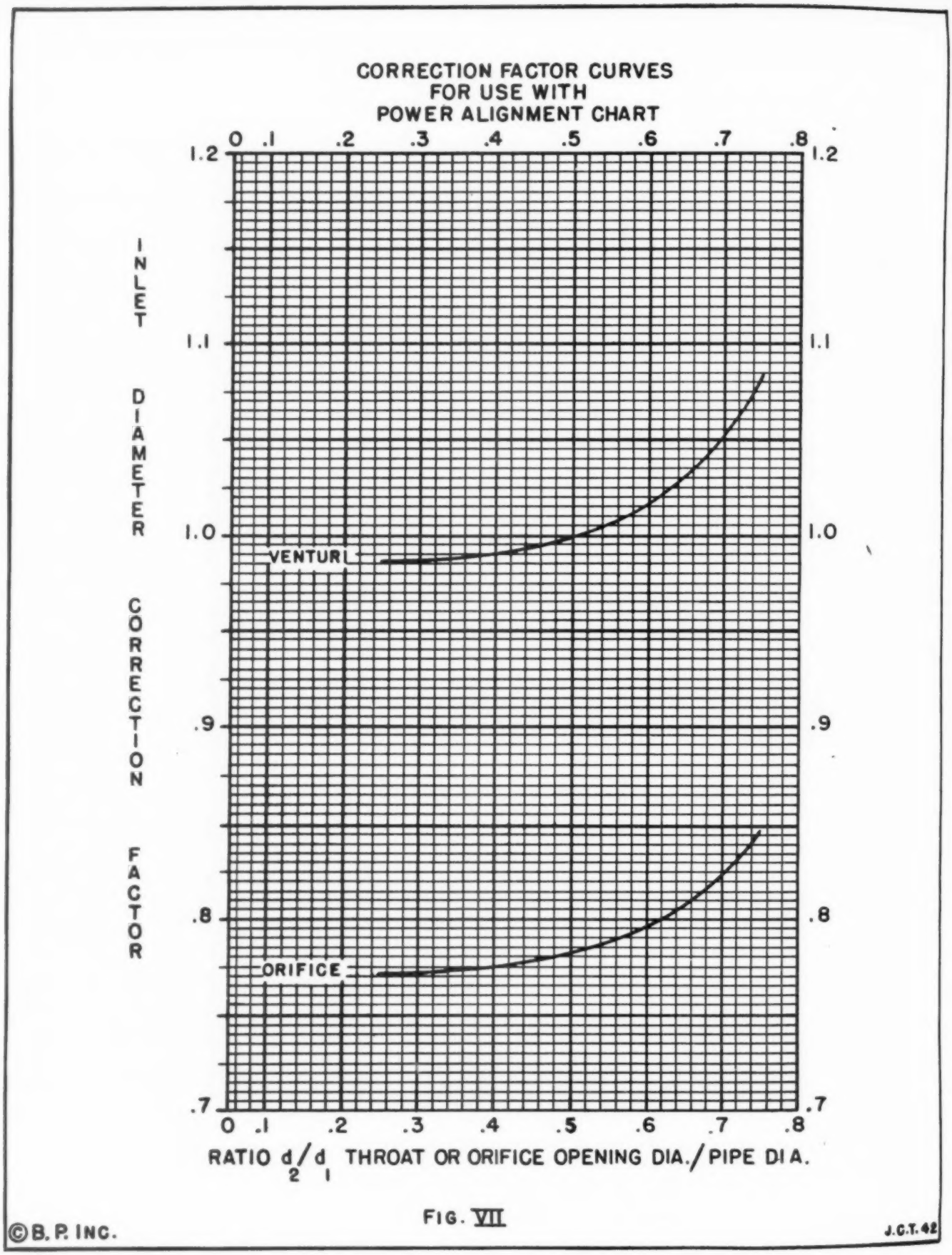


Fig. VII—Correction Factor Curves. (For use with Alignment Chart for Power Losses—Fig. VI.)

Conclusion: On the basis of a saving of power of only $2.58 per year, it is readily apparent that it will take 36 years to pay the increased cost of the Venturi Tube, and therefore, from a monetary viewpoint, the Orifice Plate should be selected.

To simplify the calculations needed for any meter installation a Horsepower Alignment Chart (Fig. VI) has been prepared. It is easily used and the results are accurate to within a few per cent. Its application to the two problems calculated in the preceding paragraphs is illustrated:

Problem A

16 inch × 6½ inch Venturi Tube compared with 16 inch Orifice Plate having 8.25 inch opening.

Flow Rate, 3,700 G.P.M.

(1) For the Venturi Tube $d_2/d_1 = \frac{6.5}{16.0} = .406$

For the Orifice Plate $d_2/d_1 = \frac{8.25}{16.0} = .515$

(2) From Fig. VII, "Correction Factor" for the Venturi Tube is .992; for the Orifice Plate .783.

(3) To obtain the "Inlet Diameter Corrected for Ratio" for the Venturi Tube or Orifice, multiply the throat diameter or Orifice opening by correction factor × 2. For the Venturi Tube, 6.2 × .992 × 2 = 12.89. For the Orifice Plate 8.23 × .78 × 2 = 12.88. Theoretically, these two products should be exactly the same; their close agreement checks the accuracy of the work.

(4) From the point 12.89 on the Corrected Inlet Diameter Scale (I) draw a straight line through 3,700 G.P.M. on Rate of Flow Scale II. Extend this line until it intersects the Index Line.

(5) From the "Recovery Characteristics" Chart, Fig. IV, the friction loss of a Venturi Tube with d_2/d_1 ratio of .406 is 11.5% of differential; for the Orifice Plate with d_2/d_1 ratio of .515 it is 71%.

(6) From the point previously located on Index Line III of Alignment Chart, draw two lines, one to the 11.5% point on the Loss of Head Per Cent of Differential Scale IV; the other to the 71% point on the same scale.

(7) These two lines cross the Horsepower Scale V and the points of intersection represent the horsepower loss for the Venturi Tube and Orifice, respectively 2.15 and 13.25.

(8) The difference in cost of operation of the two differential producers can be quickly determined by using the following simplified formula:

Excess cost of using Orifice Plate over Venturi Tube

$$= \frac{(HP_o - HP_v) \times K \times T \times C}{E}$$

In which:

HP_o = Horsepower Loss for Orifice Plate

HP_v = Horsepower Loss for Venturi Tube

K = Conversion Factor, HP to KW = .746

T = Total hours of operation

C = Cost of Power per KWH

E = Wire to water efficiency of pumping unit (motor and pump) usually 50% to 80%.

Substituting the values in Problem A

Excess cost of Orifice

$$= \frac{(13.25 - 2.15) \times .746 \times (3600 \times 15) \times .015}{.70} = \$962.00$$

This corresponds almost exactly with the previously calculated difference in cost of $1,145 — $185 = $960.00.

Problem B

4.0 inch × 2.0 inch Venturi Tube compared with Orifice having 2.49 inch opening. Flow rate 100 G.P.M.

(1) For the Venturi Tube $d_2/d_1 = \frac{2.0}{4.0} = 0.5.$

For the Orifice

$$d_2/d_1 = \frac{2.49}{4.00} = .622$$

(2) From Correction Factor Chart (Fig. VII) correction factor for Venturi Tube is 1.00 and for Orifice Plate .80.

(3) Inlet diameter corrected for ratio: Venturi Tube 2.0 × 1.00 × 2 = 4; Orifice Plate 2.49 × .80 × 2 = 4.

(4) From point 4 on Corrected Inlet Diameter Scale I, draw a line through the 100 mark on the Rate of Flow Scale II. Extend this to the Index Line III.

(5) From Chart, Fig. IV, friction loss of a Venturi Tube with d_2/d_1 ratio of .50 is 10½% of the differential; for an Orifice Plate with a d_2/d_1 ratio of .622 it is 60.5%.

(6) From the point previously located on Index Line III, of Alignment Chart, draw two lines, one to the 10.5% point on the loss of Head Per Cent of Differential Scale IV and the other to the 60.5% point.

(7) The points of intersection of these two lines with the Horsepower Scale represent the horsepower loss for the Venturi Tube and Orifice, respectively, .0038 and .024.

(8) Difference in cost of operation

$$= \frac{(HP_o - HP_v) \times K \times T \times C}{E}$$

$$= \frac{(.024 - .0039) \times .746 \times 5250 \times .02}{.60} = \$2.63$$

This solution by use of the diagrams agrees very closely with the calculated figure of $2.60.

Following the procedure as outlined and with the aid of the several diagrams presented anyone, after a little preliminary practice, can by their use quickly determine the most economical type of meter for any installation under consideration.

R. W. SPARLING

MANUFACTURER OF

MAIN-LINE WATER MEASURING EQUIPMENT

Box 3277, Terminal Annex, LOS ANGELES 54 — 3104 S. Michigan Avenue, CHICAGO 16
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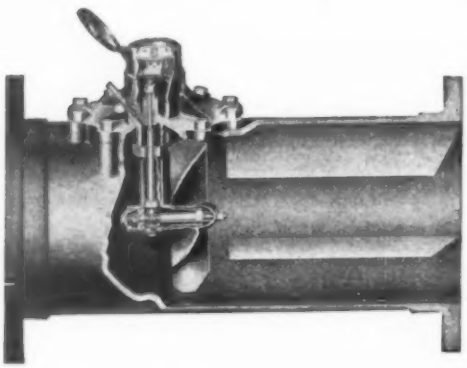


TOTALIZATION is the primary step in all Sparling Main Line Meters. Flow-rate indicators, recorders and controls are added as desired. Accuracy at rated ranges is guaranteed within 2 per cent, and loss of head is practically nothing!

FLANGED TUBES—2-inch to 36-inch
SCREW TUBES—2-inch to 6-inch
BELL & SPIGOT—4-inch to 36-inch
SADDLE-MOUNTED—6-inch to 72-inch

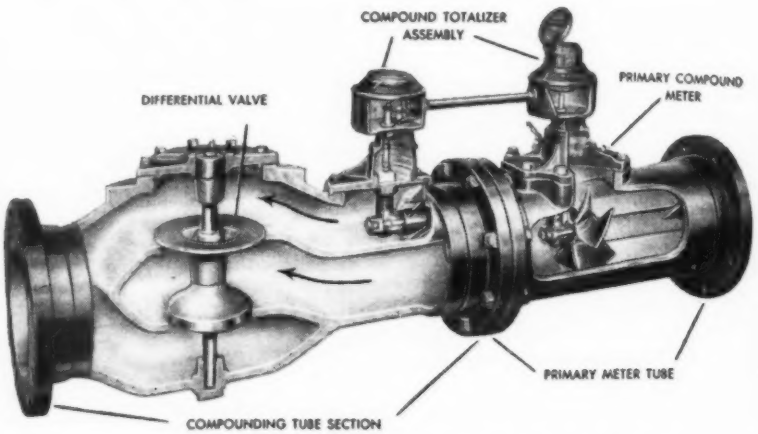
SPARLING Meters are installed as easily as a length of the pipe itself. They can be mounted on the suction or the discharge side of pump.

The Compound meters are installed horizontally, but the tube and saddle type Sparling Meters may be installed horizontally, vertically, or on an angle.

Registration is in any quantity unit desired.

Fire Hydrant Meter

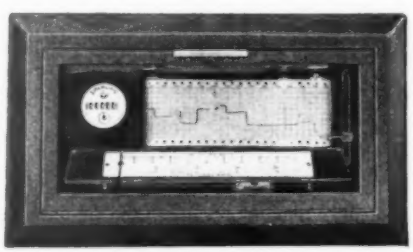
Easily attached to the hydrant and connected to fire hose for accurate measurement of flows from 30 gpm up, with only 1 pound per square inch pressure loss at 200 gpm. Weight, 20 pounds. With Standard Hose Coupling (2½"), $200.



COMPOUND Meters for Mains 6-inch to 36-inch

Consistent accuracy over flow-range as wide as 1-to-125

Differential valve cracks at about one pound per square inch back pressure from the smaller meter. The smaller meter measures all flows below the minimum accurate range of the larger meter. Both meters drive through ratchet clutches to a common totalizer, which thus operates with the faster turning meter.



INDICATORS • RECORDERS
CONTROLS

Flow-rate indicators and recorders are available for use with Sparling Meters if, as and when required. The mercury-governor type, with circular charts for 24-hour or 7-day records, are mechanically controlled within 15 feet of the meter, or electrically controlled at any distance. They may be Pedestal, Wall or Panel mounted.

The differential type, shown above, has a straight-reading 60-day strip roll chart. Indicator scale is calibrated in two flow equivalents. Operated by electric remote control. Wall, Panel or Pedestal mounted.

Mechanical or electric controls for autometered chemical feed are available.

Catalog-Bulletin 310 comes upon request

PROPELLER-TYPE METERS

For Main-Line Metering

JAMES M. MONTGOMERY*
Consulting Engineer, Los Angeles, Calif.

THE principle of the current meter has been used as a primary metering element for the measurement of stream flows for over thirty years. During the past fifteen years this same current meter principle has been applied as a primary element for the measurement of main-line flows by R. W. Sparling, of Los Angeles, and has proven itself well suited for this purpose.

The Propeller-Meter

The propeller-type meter, as illustrated in Figure I, is made up of a six-bladed propeller approximately eight-tenths of the pipe diameter mounted transversely in the center of the pipe. The propeller is made of a plastic material, having a specific gravity of approximately one, and is suitable for use in either cold water or hot water up to 300° F. The propeller practically displaces its own weight in water and is mounted on ball bearings which allow it to rotate freely in the stream of water.

Calibration

Experiments and experience have shown that a propeller will rotate in direct proportion to the velocity of water in a pipe at all flows above an approximate pipe velocity of one-half linear foot per second. The index, or the number of cubic feet passing through the pipe for each turn of the meter shaft, is determined from volumetric tests. From this index proper gears are furnished for direct totalization of the flow.

Characteristics

The current metering principle has certain characteristics that make it particularly well adapted for main-line measurement, as the action of the propeller is solely dependent upon the velocity of flow in the line. The pressure requirement is only such as to always maintain a full pipe of water. The loss of pressure caused by the metering element in sizes 6" and larger does not exceed six (6) inches of water at normal maximum pipe velocity.

Continued accuracy of the current type meter is dependent chiefly upon maintaining a clean metering section and a propeller free from incrustation. The plastic propeller has proven itself well suited for this purpose as it is practically inert and will resist both corrosion and electrolytic action. In order to maintain a uniform meter section the propeller assembly may be mounted in either a cast-iron or steel tube having a bronze liner.

Installation

The metering element may be installed directly on existing pipe lines by means of a saddle either bolted or welded directly on the main. In such installations it is important that the inside diameter of the pipe be accurately determined. In all installations the meterhead may be removed from the pipe line very quickly and easily and it is normally good practice in sizes 12" and smaller to make a yearly inspection and sizes 16" and larger, every five years.

In selecting the proper location for the metering element it should be kept in mind that a normal flow of water, free from either spiralling or jet action, is required for accurate results. The spiralling effect that may be caused in a flow of water passing through a series of elbows, can be largely corrected by installing straightening vanes in the line preceding the meter.

Various forms of such vanes are in use but a very simple and effective correction may be made by the installation of three vanes — each approximately one-third the pipe diameter and two and one-half pipe diameters long, set at 120 degrees to each other in the pipe. These vanes will not eliminate a disturbance caused by a partially open gate-valve, but are effective for double elbows or other standard fittings and with such vanes the meter element may be installed with but two pipe diameters of straight approach.

One pipe diameter downstream is normally sufficient for all installations. Unless approximately ten pipe diameters are available ahead of the meter, it is always good practice to use the straightening vanes. When the metering element is installed complete, with either a flange or bell and spigot tube, the three vanes mounted directly inside this metering section constitutes a complete unit ready for installation.

The metering element can be installed in either a vertical or horizontal pipe line and is suitable for mounting on either the suction or discharge side of a centrifugal pump.

Compound Design

Figure II is a sectional illustration of a compound meter suitable for use where an extremely wide range of flow measurement is required. A range of 150 to 1 is possible. A compound unit consists chiefly of two propeller-type meters working in series

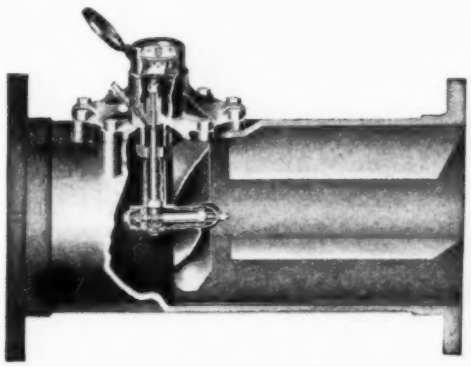


Fig. I—Cutaway View of Propeller Meter Note the flow straightening vanes in the tube, the whole unit ready for insertion as a single piece in the main-line.

through a pair of rachet clutches, allowing the meter revolving at the highest speed to drive the totalizing register. On the very low flows the large meter will under-register and thus allow the small meter to drive the totalizer. As the flows increase a pipe velocity is reached where the large meter has come into step and the small meter is operating at a high capacity. At this point the pressure loss caused by the small meter operating on the differential area of the compounding valve forces the valve to lift itself and allow the higher flow to by-pass the small meter.

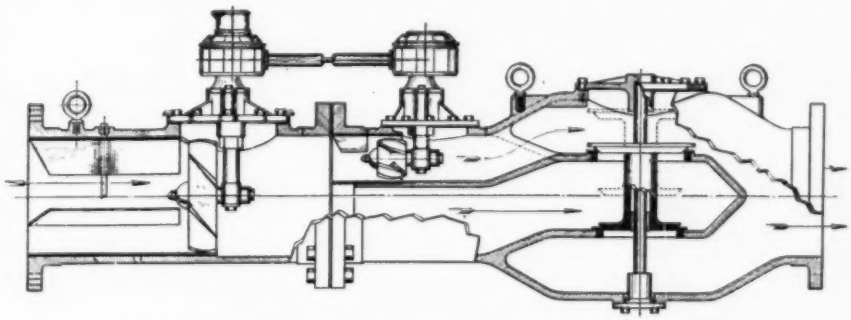


Fig. II—Section Through Propeller-Type Compound Meter Note valve which closes on low flows to cause entire flow to pass through small meter.

Although the minimum flow is considerably above that of the standard domestic compound meter it is well adapted for main-line measurement as the pressure loss caused by the entire unit does not normally exceed two pounds at maximum flow.

*President, J. M. Montgomery & Co., Engineer.

Indicating and Recording Elements

A rate-of-flow indicating and recording instrument may be used in conjunction with the straight totalizing primary element. The primary step of a current type meter is direct totalization. If the rate-of-flow indication is desired it must be obtained through a secondary step normally involving either a governor bowl or a friction plate revolving at a constant speed with a follower locating itself on the plate by impulses from the primary element.

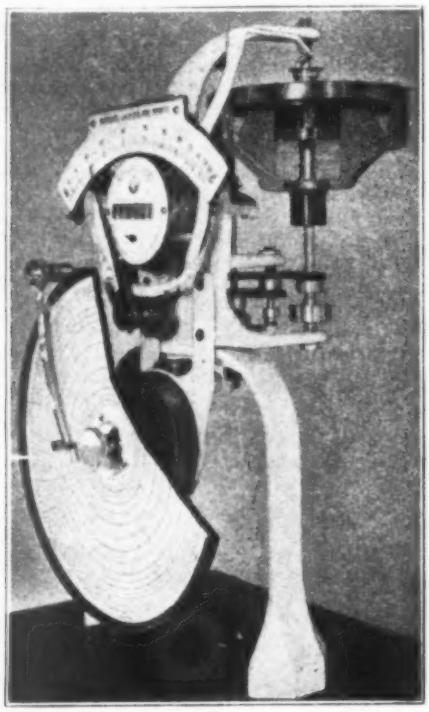


Fig. III—Typical Sparling Indicator-Recorder of the Mercury Governor Type.

Figure III illustrates the recording instrument involving a governor bowl which is driven either mechanically up to 15 feet from the metering element, or electrically to any distance required. The governor bowl consists chiefly of a float riding in the center of a mercury well. As the governor revolves, the mercury, through centrifugal force, finds its way through a small needle-valve to the outer chamber of the bowl, allowing the float in the center to have a rising and falling action in proportion to the revolving speed of the bowl. This type of governor is effective over approximately a 6-to-1 range and is extremely simple and well suited for normal pump measurement.

A recording instrument utilizing the revolving friction plate, with a follower locating itself by impulses from the metering element, as mentioned above, produces a straight line of rate indication over approximately 25-to-1 range and is well suited for recording flows of main distribution or consumption lines.

Recorders can be adapted to show the combined flows of two or more pipe lines and combinations may be furnished showing the net total of several lines where the flow of any one pipe may be in either forward or reverse direction.

Application in Chemical Feed Control

Automatic control of chemical feeders by either mechanical or electrical means is readily obtainable from a current meter element as the shaft of the meter is revolving in direct proportion to the flow and considerable load may be applied without affecting the rotating speed of the propeller.

If given the period inspection required by any primary metering element, the characteristic of low pressure loss and accuracy over a wide range of flow allows the principle of the current meter to fill an important part in the waterworks field.

The propeller-type meter is not suitable for measuring raw sewage and is not generally recommended for measurement of a sewage effluent unless it has passed through a filter bed. Care must also be taken when installing such meters on raw water gravity lines that screens are provided at the inlet structure to prevent excessive algae or weeds from passing down the pipe and stalling the meter propeller.

WHERE THE WATER DOLLAR GOES

Here is an interesting and enlightening chart taken from the 22nd Annual Report of the Water Department, Springfield, Ill.

It is based on a survey made by H. V. Pedersen, Water Supt., at Marshalltown, Iowa, to determine what percentage of water in the average consumer's home goes for various uses.

The Springfield folks have taken these percentages and made them more real to the customer, by showing him that out of every dollar paid for water used it only cost him 1 cent to wash his car, only 4 cents for laundry work at home, 3 cents for sprinkling in summer.

R-S Products Corporation

Valve Division, Wayne Junction, Philadelphia 44, Pa.

R-S Valves for Precise Control and Shut-off

Distributors in the following cities:

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The Disc or Butterfly type of valve, properly designed and constructed, has numerous advantages.

Minimum Pressure Drop saves Power

The bevelled disc of the R-S Valve seats firmly against the body when fully closed. Perfect machining reduces leakage to a minimum with metal-to-metal seats. Drop-tight closure may be had with R-S Rubber Seated or Rubber Lined Valve.

In the open position the streamlined disc creates a Venturi action.

Pressure-drop, as compared with many other types of valves, is low, and the R-S Valve therefore shows substantial savings in pumping power. In many cases the saving is tremendous.

Regulation and Closure are Quick

R-S hand-controlled valves are fully closed from full open position by the movement of a lever through 77½° of arc. Either lever or wheel may also be actuated by a chain.

Power-controlled prime movers delivering 15 foot-pounds to 38,000 foot-pounds of torque open or close at any desired speed. Full closure takes 1 second to 10 minutes depending on the type of material controlled and desired operation.

Uniform Control in all Positions

R-S Valves are unique in giving uniform control of flow over most of the arc through which the disc travels from full open to full closed position.

More Compact, Lighter, Lower in Cost

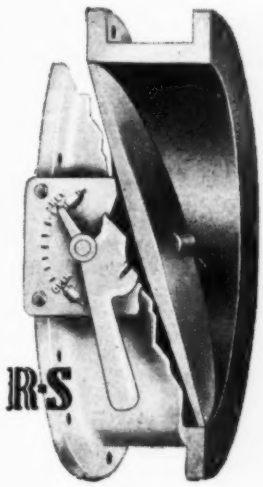
R-S Valve design gives a narrower face-to-face than most types of design. The valve is lighter and requires less support. Less metal is required for its construction; therefore less machining, resulting in lower cost.

Simpler . . . Self-cleaning . . . Non-turbulence Reduces Wear

R-S Valve design offers no pockets to capture sediment, no change of flow-direction to create turbulence. Freedom from turbulence reduces erosion and cavitation and therefore adds to the life of the valve.

A Principle

Every R-S Valve is designed and built for extremely rugged service, and provided with such safety-factors that it will give complete satisfaction in the service for which it is designed.



USES

Shut-off and regulation of volume and pressure
Pressure relief
Control of:
- liquid level
- back pressure
- water hammer
- steam hammer
- constant differential pressure
- output of pumps, fans, engines, turbines
- automatic combustion

FOR;

Air
Gas
Liquids
Steam
Semi-solids

TEMPERATURE RANGE

—300° F. to plus 2000° F.

THREADS

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American Sanitary

SIZES

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American Standard
Spiral Pipe Mfrs.
and a large variety of special flanges

PRESSURES

2 to 2500 psi.

TYPES OF CONTROL

Hand lever
Chain lever
Hand wheel
Chain wheel
Solenoid trip
Float
Electric Motor
Air Diaphragm Motor
Hydraulic Cylinder
Air Cylinder
Automatic control can be furnished with positioner if desired

TYPES OF METAL

Aluminum
Brass
Bronze
Cast Iron
Meehanite
Cast Steel
Monel Metal
R-S "A" Metal (anti-abrasive)
R-S "H" Metal (heat-resisting)
Stainless Steel or any other material that can be cast or welded

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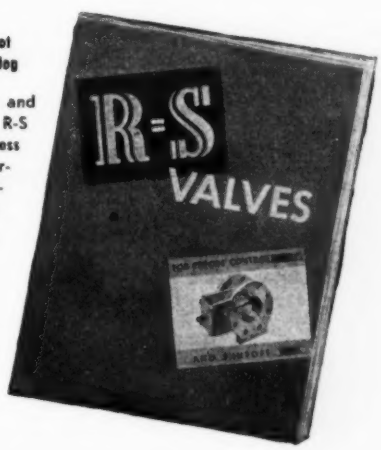
Full descriptions and specifications in R-S Catalog 17. Address R-S Products Corporation, Valve Division, Wayne Junction, Philadelphia 44, Pa.



ACCOUNTED-FOR WATER AND UNBILLED REVENUE

A Method for Estimating Both Between Scheduled Meter Readings

By CHAS. E. MOORE
General Manager, Roanoke Water Department, Roanoke, Va.

THE war brought forcibly to the attention of the home front the need of conservation of the critical materials.

The water department's part in that program may be generally divided into two main features. The first requires that it place its plant in the best over-all condition, the main point of which is to account for all of the water taken from its source of supply and put into its distribution system.

Where communities are served by an unlimited gravity supply requiring no treatment, conservation plays a very minor part. On the other hand if supply is treated chemically and is also pumped, reducing the volume introduced into the distribution system to an economic level, produces a saving in chemicals, in fuel, and by their reduction, a reduction in the need for transportation facilities.

The other feature of the water department's part in the conservation program is to educate its consumers to be as conservative in their use of water as is possible.

That water department, which renders its service on a flat rate basis, has no exact measure of the use of its service made by individual customers. It has a peculiar problem that will not be answered by this article which is directed to those plants where practically all customers are metered.

Basic Assumptions

The practice of billing all customers monthly is rapidly being eliminated, and meter readings are now being taken at longer intervals than heretofore. For the purpose of discussion, we may assume that quarterly meter reading has been adopted and, further, that in the interest of economic commercial work one-third of the total customers are billed each month. Two further assumptions are made: That the individual meters serving the customers are read all during the month, and that bills resulting from the meter readings are made and delivered within two or three days after the meters are read.

Such practice makes the determination of the quantity of water delivered to the customer, as compared with the total quantity pumped, something of a problem. This article undertakes to solve that problem by determining what percentage of the pumpage is actually sold.

The foregoing meter reading practice produces a consumption figure for one-third of the city, closely approximating the total quantity sold for a three-months' period for that entire section as of the middle of the month.

During any quarter period the other two sections have already been billed for a part consumed in the quarter and have also received service of an undetermined quantity for the balance of this quarter. The problem of determining the quantity delivered to all sections, and compared with the quantity of water pumped as of the last day of the month, now presents itself. This article submits for consideration the reconciliation between the water pumped and the water billed by a method of graphs.

Preparation of Chart for Accounted-for Water

Using cross-section paper with a convenient scale, lay out on a vertical axis the total gallons to be pumped for the 14½ months' period beginning with October 15th; on the horizontal scale mark off an equal space representing the 14½ months. The variation in the number of days in the different months will be found to be insignificant and of no real purport. The accompanying figure shows the chart.

At the top of this graph prepare a table of eleven columns as shown. Col. 1 contains the names of the months from December through December inclusive; i. e., 13 months. Column 2 provides for the actual known deliveries of Section 1 for each quarter of the year. Column 3 provides for an accumulative figure of Section 1 to date. Columns 4, 5, 6, and 7 provide for similar figures of Sections 2 and 3 respectively. Column 8 provides for the actual quantity pumped for each month; column 9, the accumulated pumpage. Column 10 provides for the estimate of the water delivered to the customer as taken from the graph; and column 11 is for the determination of the percentage ratio between water accounted for and the water pumped.

Step 1. The first step in preparing the graph is to make an estimate of the quantity of water used by 1/3 of the consumers (i. e., Sect. 1) in the previous quarter as determined from meter readings taken during the month of January. This value is given in Col. 2. Consider that amount as billed the customer as of the middle of January. This is point 1 on the graph.

This estimate is, of course, based on past history. On the graph indicate this quantity by a point half way between the December and the January line.

Step 2. Make a similar determination of the quantities that will be billed for Section 2 during the month of February, and then the third section during the month of March. These are points 2 and 3 respectively, and are the data values given in cols. 4 and 6.

Step 3. Draw on the graph a portion of a line beginning in time three months prior to each point (1, 2, and 3) at zero, and draw the three lines to the end of December only. See heavy lines on graph. It is suggested for convenience that these three lines be drawn in three colors.

Step 4. Add the ordinates of the three lines where they cross the December line and indicate their sum at the end of December by a point, "Est." This ordinate represents your estimate of the water actually delivered as of December 31st. Enter this estimate in the table opposite the top December under Col. 10 "Acct. For."

Step 5. With little or no previous data, you must now assume a factor of accounted-for water and enter on December line under Col. 11. Apply this factor to the "Acct. For" estimate (Col. 10), determine the "Gross" water that has been pumped corresponding to this estimate of water delivered, and enter this pumpage figure on the December line under Col. 9 "Gross."

Step 6. At the end of January, record on the January line in Col. 3 the pumpage for the month of January. Add this value to the Dec. "Gross" in Col. 9 above, and enter the sum on the January line under Col. 9, Gross.

Step 7. When the January billings have actually been determined and corrected to the known unmetered usage, plot this corrected figure half way between December and January (i. e., Jan. 15) and redraw the correct line from zero as of the 15th of October through this point, and project the line to January 15 for point a. Next project the curve on the graph for Sections 2 and 3 to January 31 in the same manner, thus obtaining points b and c on the graph.

Step 8. Now add the three ordinates of the three sections as of January 31 and record on the graph their total, point "A." Reading from the graph, insert this figure in Col. 10, Accounted-For. This value with the Jan. value in Col. 9 will permit a new determination of the percentage accounted-for to be entered in Col. 11.

Step 9. The process is repeated in subsequent months, and each quarter the estimate previously made is corrected to the known facts. With this initial very close determination of the percentage accounted-for, the succeeding years' graphs will be correspondingly more accurate.

As each month passes, points 4, 5, 6, then 7, 8, 9, etc., will be definitely determined, with points d, e, and f, then g, h, and i, representing the estimates at the end of each month; the latter groups adding up to B and C, and so on for the year.

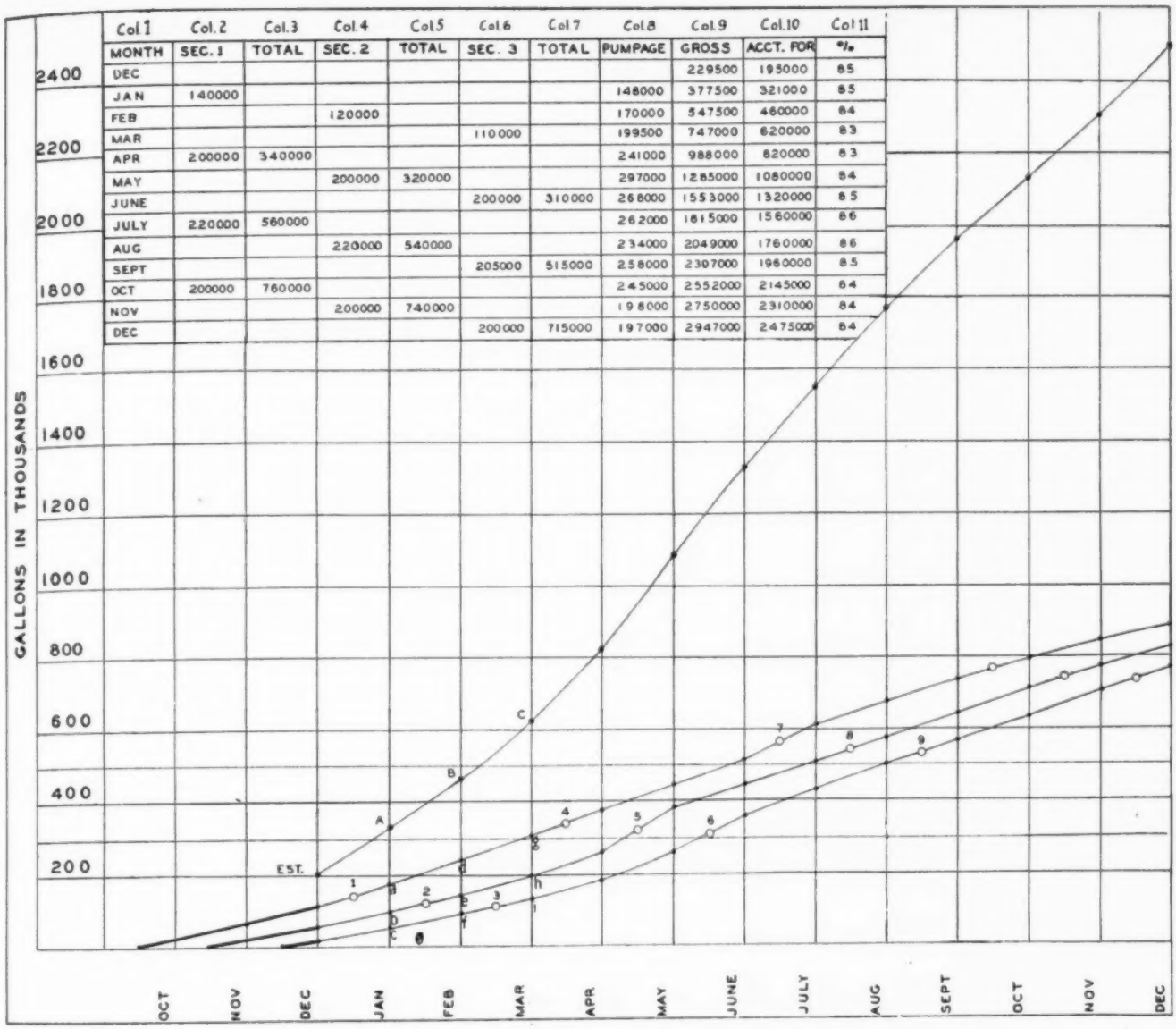
It is to be remembered that, while this graph is for the purpose of determining the leakage in the distribution system, it will also include the loss due to under-registration of meters.

Unbilled Revenue

Service to consumers is rendered and costs paid before consumer is billed. Modern accounting practice requires an estimate of the revenue to be derived by a future billing of service already rendered.

The determination of unbilled revenue follows exactly the same fundamental scheme as outlined above. Again use a table of eleven columns. In Cols. 2 to 7 incl. record the regular quarterly billings and the accumulation by quarters. In the subsequent columns, Col. 8, 9, 10, are the accumulated gross billings, the projected revenue, and the difference between the two; that is, unbilled revenue. Col. 11 will show the increase or decrease in this unbilled revenue. (Note, no graph or chart appears herewith for this Unbilled Revenue calculation.)

The curve is started in the same manner as outlined in Step 1 above. An estimate of the revenue for the first three quarters is made, thus determining as of December 31 that portion al-



Col 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11
MONTH	SEC. 1	TOTAL	SEC. 2	TOTAL	SEC. 3	TOTAL	PUMPAGE	GROSS	ACCT. FOR	%
DEC								229500	195000	85
JAN	140000						148000	377500	321000	85
FEB			120000				170000	547500	460000	84
MAR					110000		199500	747000	620000	83
APR	200000	340000					241000	988000	820000	83
MAY			200000	320000			297000	1285000	1080000	84
JUNE					200000	310000	268000	1553000	1320000	85
JULY	220000	560000					262000	1815000	1560000	86
AUG			220000	540000			234000	2049000	1760000	86
SEPT					205000	515000	258000	2307000	1960000	85
OCT	200000	760000					245000	2552000	2145000	84
NOV			200000	740000			198000	2750000	2310000	84
DEC					200000	715000	197000	2947000	2475000	84

Chart and Table for Determining Accounted-For Water

(See steps 1-9 in accompanying text for details of construction—the chart for unbilled revenue is developed in the same manner.)

ready delivered for the three sections of the city not yet billed. This is, of course, the unbilled revenue as of December 31 and also equals the projected revenue as of this date.

When monthly billings are known, add the accumulated gross billings in Col. 8, complete the graphs to end of the month in the same manner and upon the same theory as used in the pumpage figures.

The projected revenue determined by the summation of the three alternates is read off the graph and entered in Col. 9. The difference of the two is the new unbilled revenue figure to be entered in Col. 10 with the increase or decrease as the case may be entered in Col. 11. This last column should begin with the low pumpage period, increase to a peak in the high consumption period, and then decrease to approximately its starting point, other general business conditions remaining constant.

It will be found that the time required to prepare these graphs, once they are started and understood, is insignificant. On the other hand the management is possessed of a very accurate determination of the accounted-for water and the relationship between revenue and expense; for, under this quarterly billing scheme and a uniform rate of reading meters, expense of operation will tend to run ahead of actually billed revenue.

HORSE-POWER AND ELECTRICAL DATA

(a) 1 H.P. = 0.746 KW. = 746 watts.

(b) Brake H.P. $= \dfrac{\text{KW. input}}{0.746} \times$ motor efficiency.

(c) Water H.P. $= \dfrac{\text{g.p.m.} \times \text{head in ft.}}{3960}$.

(d) H.P. to lift water $= \dfrac{\text{g.p.m.} \times \text{head in ft.}}{3960 \times \text{pump efficiency.}}$

RATING AND SAMPLING WELLS BY AN IMPROVED METHOD

By PAUL F. HOWARD, M.S.*

Consulting Engr., Boston, Mass.

It is oft-times desired and of importance to ascertain the quantity and quality of water yielded by an individual driven well connected to other wells of a water system. Experience has shown that analyses of samples of water taken from individual wells by means of a hand pitcher-pump give false and meaningless results, especially in relation to the iron content. Likewise the yield of an individual well by means of separate pumping furnishes but little if any information as to the yield of the individual well when ganged together into a common system.

The writer's firm was engaged to investigate a water supply from about 90 driven wells. During the course of the work it became desirable to determine the yield of each individual well and the iron content of the water from each individual well under normal operating conditions. This was done by a device which we have called a Well Rater and Sampler. The device (see cut) was connected to the top of the well by a 2½ inch wrought iron pipe about 2½ feet long. At this point a 1¼ inch centrifugal pump was connected by the proper fittings and discharged through a 2 inch pipe. On the suction pipe a hand pump was attached to prime the pump. A 2 inch orifice flow meter was inserted in the discharge line and by means of a manometer the rate of flow was determined. The amount of the draw-down of the well, when connected to the well field, was determined by means of a mercury U tube connected to a small tube attached to the well cap, the tube extending into the well to a depth sufficiently great so that the end was at all times below the level of the water-table outside of the well.

Before the Well Rater and Sampler was connected to the well the permanently installed valve on the suction line was closed. After the device was connected this valve was opened and the well was observed under actual working conditions—i.e., while it was being pumped along with the other wells by the large pump in the pumping station. The drawdown was revealed by the reading of the manometer connected to the tube in the well and recorded. Thereafter, the permanently installed valve was closed and the centrifugal pump started, the rate of pumping being controlled by a valve on the discharge line. As the discharge increased the drawdown increased, thus producing a deflection on the manometer. When the manometer reading was the same as that already observed under the actual working conditions, as just described, the output of the well was assumed to be the same. The quantity being pumped was then determined by observing the deflection of the flow manometer on the pump discharge line. The results represented the actual amount of water that the well was contributing from the field as a whole when under *actual* pumping conditions. Samples of water for analyses, then taken at the point of discharge, were considered truly representative samples.

By careful operation of the valve at the discharge end of the pipe the yield of the well was brought gradually up to the original yield at the beginning of pumping. Thereby higher velocities than usual or ordinary in the pipe well were provided. Any excess velocities would, of course, free scale, rust and other matter and give false samples. The flow manometer in this particular instance was connected to the 2 inch orifice flow meter. [The liquid used to determine the differential pressure can be of any specific gravity desired depending upon the rate of flow to be measured. The most common is mercury for high flows and carbon tetrachloride, Specific Gravity 1.60, for low flows.]

One side of the mercury U tube was connected to the tube in the well and filled with water from the surface of the mercury to the water level in the well, care being taken to remove all air. The other side of the U tube was left open to the atmosphere.

*Whitman and Howard, Engs., Boston, Mass.

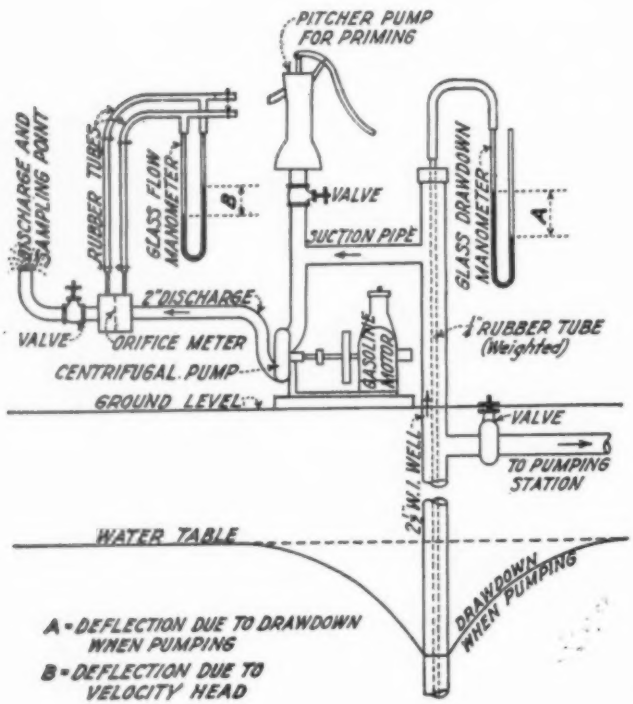
A Specific Case

The following specific example will serve as an illustration. After the Well Rater and Sampler had been properly set up and connected to a well the mercury U tube was connected to the tube in the well. There was an initial deflection of mercury of 3 inches which represented a water column of 40½ inches, the distance from the water table to the mercury surface of the U tube. After an equilibrium had been reached the well was connected to the field and the mercury in the U tube deflected an additional amount corresponding in general to the distance the water table dropped outside of the well and specifically to the drop in pressure in the well. In this case the total deflection was 4¼ inches or a difference in deflection of 1¼ inches which equal 17 inches of water as the drawn down within the well.

The well was then shut off from the other wells and the small test pump started. The discharge valve on the pump was then opened slowly, increasing the yield of the well until the drawdown deflection of mercury showed 4¼ inches. After this point was reached the flow manometer was observed to have a deflection of 1⅝ inches of mercury which on this meter represented a flow of 20.1 gallons per minute.

The yield of the various wells in relation to the drawdown was plotted on arithmetic paper and the yield of each well was found to be directly proportional to its drawdown. The yield of the various wells varied from 1 to 25 gallons per minute and the drawdown varied from 5½ to 26 inches.

This device has been made use of in installing new driven well fields. By its use we have been able to predict reasonably well the draw down or vacuum to be expected in a system of driven wells and to eliminate uneconomic wells, which have a low yield in relation to the drawdown. Also with open end wells it has been possible to determine at what yield a well will throw sand sufficiently to result in filling up by sand. The equipment will probably be found to have other uses also.



WELL RATER AND SAMPLER

The Hook-Up of the Well Rater and Sampler

PITTSBURGH-DES MOINES STEEL CO.

Tanks—Plate Fabrication—Structural Steel

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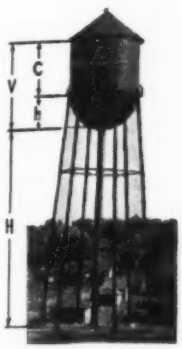
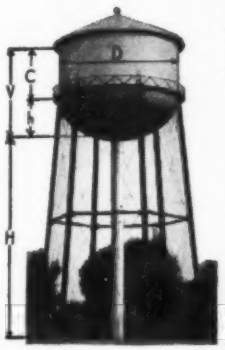
In these tables we have compiled—in compact but complete form—data on steel reservoirs and on the various types of elevated steel tanks. For information as to the application of these various types of tanks, for designs for specifications, or for estimates—write our nearest office.

Table I—Pittsburgh-Des Moines Standard Hemispherical Bottom Elevated Tanks



Capacity in U.S. Gallons	Tank Dimensions D ft.	C ft. in.	V ft. in.	H = Height of Tower to Bottom of Tank Capacity											
				ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.
5,000	9	10 0	11 3	11 3	18 9	31 9	38 3	51 9	58 9	72 9	79 9	93 9	100 9	.....	.....
10,000	13	5 11	11 11	10 0	20 0	30 0	40 0	50 0	60 0	70 0	80 0	90 0	100 0	.....	.....
15,000	13	10 11	16 11	10 0	20 0	30 0	40 0	50 0	60 0	70 0	80 0	90 0	100 0	.....	.....
20,000	13	15 11	21 11	10 0	20 0	30 0	40 0	50 0	60 0	70 0	80 0	90 0	100 0	.....	.....
25,000	15	14 7	19 11	44 8	50 0	69 8	75 0	.....	94 8	100 0	.....	125 0	.....	.....	.....
30,000	15	18 5	23 9	44 8	50 0	69 8	75 0	.....	94 8	100 0	.....	125 0	.....	.....	.....
40,000	17	18 5	25 1	46 8	.....	.....	75 0	.....	96 3	100 0	.....	125 0	.....	.....	.....
50,000	19	17 7	25 7	.....	55 0	63 4	75 0	83 4	91 8	100 0	111 8	120 0	128 4	140 0	148 4
60,000	19	22 6	30 6	.....	55 0	63 4	75 0	83 4	91 8	100 0	111 8	120 0	128 4	140 0	148 4
75,000	21	22 6	30 6	.....	55 0	63 4	75 0	83 4	91 8	100 0	111 8	120 0	128 4	140 0	148 4
100,000	24	22 6	31 0	.....	50 0	63 1	75 0	79 7	91 6	100 0	113 1	116 6	125 0	141 6	153 4
150,000	28	24 2	34 7	46 6	54 10	63 2	75 0	83 4	91 8	100 0	111 10	120 2	128 6	140 3	148 7
200,000	32	23 4	36 1	45 9	54 1	62 5	75 0	83 4	91 8	100 0	112 7	120 11	129 3	141 10	150 2
250,000	32	31 4	45 4	45 4	53 8	62 0	75 0	83 4	91 8	100 0	113 0	121 4	129 8	142 8	151 0
300,000	36	28 4	42 5	45 4	53 8	62 0	75 0	83 4	91 8	100 0	113 0	121 4	129 8	142 9	151 1
400,000	40	30 0	46 6	44 6	52 10	61 2	75 0	83 4	91 8	100 0	113 10	122 2	130 6	138 10	152 8
500,000	44	29 9	49 2	43 6	51 10	60 2	75 0	83 4	91 8	100 0	114 10	123 2	131 6	139 10	154 7
600,000	44	38 6	58 2	} Tower heights have not been standardized for these capacities and can therefore be built to suit the requirements in each particular case.											
750,000	50	34 9	57 5												
1,000,000	50	52 0	73 9												

Table II—Pittsburgh-Des Moines Standard Hemi-ellipsoidal Bottom Elevated Tanks

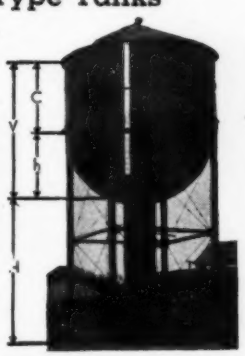


Capacity in U.S. Gallons	Tank Dimensions D ft.	C ft. in.	V ft. in.	H = Height of Tower to Bottom of Tank Capacity											
				ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.	ft. in.
25,000	17	11 9	15 5	46 4	51 8	71 4	75 0	.....	96 4	100 0	.....	126 8	...	.....	.....
30,000	17	14 7	18 6	46 1	51 5	71 1	75 0	.....	96 1	100 0	.....	126 5	.....	.....	.....
40,000	21	11 3	18 9	47 5	.....	.....	75 0	.....	97 5	100 0	.....	126 2	.....	.....	.....
50,000	22	13 2	19 0	.....	57 2	65 6	75 0	85 6	93 10	100 0	113 10	122 2	130 6	142 2	150 6
60,000	24	12 10	19 4	.....	56 6	64 10	75 0	84 8	93 2	100 0	113 2	121 6	129 10	141 6	149 10
75,000	26	13 6	20 8	.....	55 10	64 2	75 0	84 2	92 6	100 0	112 6	120 10	129 2	140 10	149 2
100,000	28	16 2	23 2	.....	51 6	64 7	75 0	81 1	93 0	100 0	114 7	118 0	126 6	143 0	154 10
150,000	34	14 10	25 0	46 9	55 1	63 5	75 0	83 7	91 11	100 0	112 1	120 5	128 9	140 6	148 10
200,000	38	15 5	27 0	46 11	55 3	63 7	75 0	84 6	92 0	100 0	113 9	122 1	130 5	143 0	151 4
250,000	40	18 1	30 0	47 5	55 9	64 1	75 0	85 5	93 9	100 0	115 1	123 5	131 9	140 1	153 1
300,000	40	23 11	34 0	49 4	57 8	66 0	75 0	87 4	95 8	100 0	117 0	125 4	133 8	142 0	155 0
400,000	46	22 9	34 11	48 10	57 2	65 6	75 0	87 8	96 0	100 0	118 2	126 6	134 10	143 2	157 0
500,000	50	23 9	37 0	49 8	58 0	66 4	75 0	81 2	89 6	100 0	112 8	121 1	129 4	146 0	152 8
600,000	55	21 9	39 7	} Tower heights have not been standardized for these capacities and can therefore be built to suit the requirements in each particular case.											
750,000	60	22 4	42 4												
1,000,000	60	35 0	54 0												
1,000,000	65	26 1	47 7												
1,250,000	79	17 4	43 4												
1,500,000	70	37 8	60 0												
1,500,000	79	24 0	50 0												
2,000,000	90' 3"	22 6	50 0												

Table III—Double Ellipsoidal Tanks

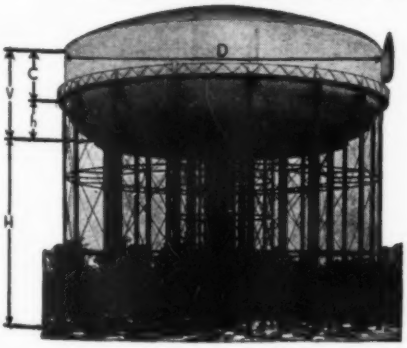
Capacity in U.S. Gallons	D	B	C	T	V	H
50,000	22'-0"	5'-10"	9'-4"	4'-4"	19'-6"	Built to Order
75,000	26'-0"	7'-0"	9'-0"	5'-0"	21'-0"	
100,000	28'-0"	7'-0"	11'-1"	5'-7"	23'-8"	
150,000	34'-0"	10'-0"	8'-8"	6'-10"	25'-6"	
200,000	36'-0"	9'-0"	12'-11"	6'-9"	28'-8"	
250,000	40'-0"	10'-4"	11'-7"	7'-6"	29'-5"	
275,000	40'-0"	11'-11"	13'-9"	7'-6"	33'-2"	
300,000	40'-0"	8'-7"	17'-11"	7'-6"	34'-0"	
350,000	44'-0"	14'-8"	13'-10"	8'-0"	36'-6"	
400,000	46'-0"	12'-3"	14'-9"	8'-8"	35'-7"	
500,000	50'-0"	15'-0"	14'-6"	9'-8"	39'-2"	
600,000	55'-0"	14'-4"	14'-0"	10'-0"	38'-4"	
750,000	60'-0"	15'-11"	12'-7"	11'-6"	40'-0"	
1,000,000	65'-0"	21'-8"	15'-9"	11'-6"	48'-5"	

Table IV—Railroad Type Tanks



Capacity in U.S. Gallons	Tank Dimensions D ft.	C ft. in.	V ft. in.	H
20,000	15	10 9	16 2	Any height from 15' 0" to 40' 0" in multiples of 5' 0"
25,000	15	14 7	19 11	
30,000	17	12 7	19 0	
40,000	19	13 1	20 6	
50,000	21	13 1	21 1	
60,000	24	10 1	20 3	
75,000	24	15 1	23 10	
100,000	28	13 5	23 7	
150,000	28	14 2	24 7	
200,000	36	15 1	29 2	
250,000	38	17 3	33 3	
300,000	38	23 11	38 3	

Table V—Toroidal Bottom Tanks



Capacity in U.S. Gallons	Tank Diameter (D) when h = 10' 0" and V = 20' 0" ft.	in.	V = 25' 0" ft.	in.	V = 30' 0" ft.	in.	V = 35' 0" ft.	in.	H
750,000	82	0	73	2	66	8	61	8	Built to Order
1,000,000	94	5	84	3	76	9	71	0	
1,250,000	105	4	94	0	85	8	79	2	
1,500,000	115	2	102	9	93	9	86	8	
2,000,000	132	8	118	5	108	0	99	11	
2,500,000	148	0	132	3	120	7	111	6	
3,000,000	162	0	144	8	131	11	122	1	
4,000,000	186	8	166	9	152	2	140	9	
5,000,000	208	6	186	3	169	11	157	3	

Table VI—Steel Reservoirs

Capacity in U.S. Gallons	Diameter ft.	Height ft.	in
50,000	20	21	6
60,000	22	21	6
75,000	24	22	6
100,000	26	25	6
150,000	30	28	9
200,000	34	29	9
250,000	38	29	9
300,000	42	29	3
400,000	46	32	6
500,000	50	34	6
750,000	60	35	9
1,000,000	70	35	0
1,500,000	80	40	3
2,000,000	95	38	3
2,500,000	110	38	6
3,000,000	120	38	9
4,000,000	130	40	9

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Est. 1854

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NEWMAN, GA.

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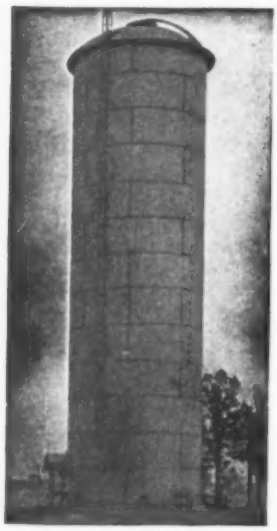

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STORAGE ON THE DISTRIBUTION SYSTEM*

BY CHARLES B. BURDICK

Cons. Engr., Alvord, Burdick & Howson
Chicago, Ill.

IT HAS been aptly said that elevated storage is like money in the bank. It represents work already accomplished and an asset that can be drawn upon for instant use when needed.

Unlike storage on any other part of the water works system, storage on the distribution system is actually beneficial to every part of the water works, extending all the way back to the source of supply, through feeder pipes, pumps, treatment facilities and, in many cases, the water development facilities.

Like money in the bank, wherein only a certain amount is necessary for day to day commercial transactions, enough is enough. Money invested in excessive storage would better be devoted to something more needed. It is the present purpose to show how much storage may be warranted under general conditions and the available means for accomplishing it, bearing in mind that no two cases are exactly alike and that while the conditions surrounding most water works are similar, there are wide variations in conditions affecting certain cases. Therefore each case should have separate study if the project is to be both technically and economically sound.

Generally speaking, that water works system is the best that provides facilities adequate to meet all conditions of service, including rare emergencies, at the least cost. The latter takes into consideration expenses of operation balanced against capital costs, usually represented by interest and depreciation upon the investment.

Storage upon the distribution system is of principal value in the form of adequacy and reliability of service, and in its economy. It is justified to the extent that it increases more constancy of pressures and greater reliability and economy, bearing in mind that in practically all installations allowance must be made for the probable service requirements for a reasonable period in the future. Both reliability and economy are promoted by the effects of storage in securing more economical pumpage, and in providing an assured pressure and adequate volume of water for fire protection.

*A basic discussion of this important topic prepared especially for the Reference and Data Number of *Water & Sewage Works* at our request.—The Editors.

Conditions Affecting Storage

Every water works man would have his problem much simplified, and the investment in water works would be much reduced, if water could be served at a uniform rate of delivery throughout the year. This is impossible, however. The consumer properly demands water when he needs it, and needs vary widely with the seasonal extremes and wide variations in demand throughout the hours of the day.

Storage is particularly beneficial in the afternoon or evening hours upon a few days in exceptionally hot and dry summers. Such summers do not come every year, and therefore excessive demand often comes as a surprise. An examination of many pumping records indicates that the demands of one summer in five or ten may greatly exceed the ordinary summer. Therefore, in providing facilities for storage it

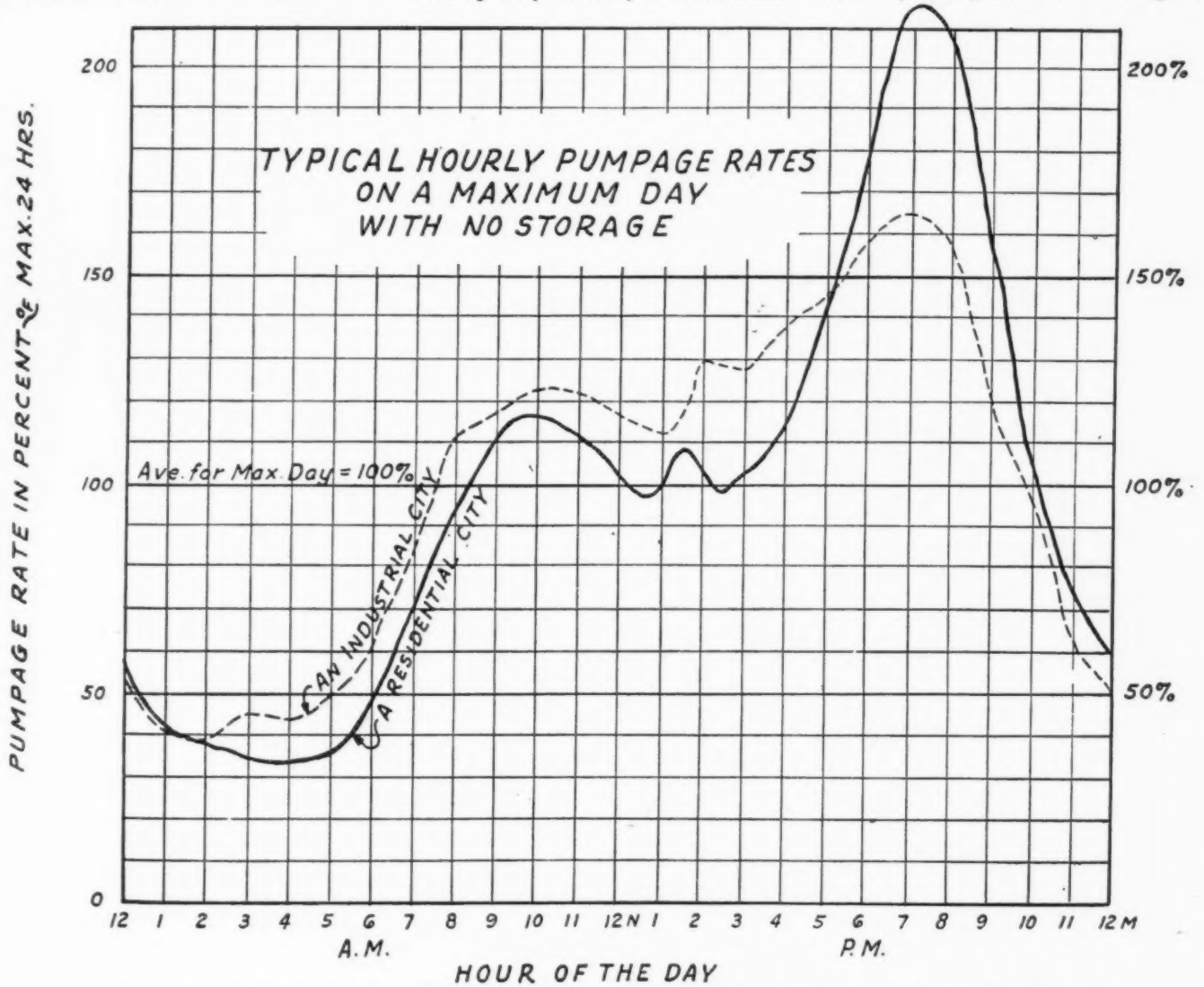

Fig. 1.—Range of Hourly Pumpage Rates on Maximum Days—Typical of Cities Without Distribution Storage. (Pattern for residential city compared with industrial city.)

is wise to allow a liberal percentage above average requirements in providing storage facilities.

The accompanying Table 1 is an analysis of the service requirements on five water works systems in cities varying from 37,000 to 320,000 population during years that are known to represent a well defined maximum in each city.

In these statistics the hourly pumpage rates were studied and correction was made for a small amount of elevated storage in several cases so that the figures would represent pumpage necessary to supply demand without storage. Fig. 1 shows diagrammatically the hourly rates of pumpage upon a maximum day for two cities, which represent common maximum and minimum conditions as to pumpage variation. There are some very large cities where use is more uniform. For more ready comparison, the pumpage rates have been expressed in percentage of the average pumpage rate (total pumpage) during the 24 hours of a maximum day. With information similar to this for any city, it is possible to compute the amount of storage which if most effectively liberated would make it possible to operate the pumps at a uniform rate throughout the 24 hours of a maximum day. In other instances, notably the small municipality, adequate storage can sometimes make possible the complete pumping requirement during the period of money-saving off-peak power rates.

Hourly variations in pumpage depend upon the character of the city in regard to similarity of use. Perhaps the most important consideration is the extent of industrial use. This can be and usually is a more or less uniform use. Upon the other hand, residential consumers use little or no water at night and make heavy demands at certain hours of the day, particularly during the early evening hours; and to this is added a very exceptional demand for lawn sprinkling on hot summer days. Recently, water used in cooling systems has placed an added burden on hot weather peak hours. In most cities the varying residential demand is more or less ironed out by commercial and industrial uses, and the extent of commercial and industrial use is probably largely accountable for differences in pumpage rates when comparing cities.

TABLE 1
Variations in Pumpage in Five Cities

Character of City	1 Residential	2 Industrial	3 Residential	4 Residential	5 Industrial
Population	37,366	100,426	110,468	307,000	320,000
Ave. pumpage MGD	3.69	9.61	6.94	17.50	43.80
Gals. per capita	99	96	63	57	137
Max. 24 hrs. MGD	5.09	17.42	13.66	30.65	70.20
Gals. per capita	136	174	124	100	220
Per cent of ave. day	138%	182%	196%	175%	160%
Max. hr. MGD	8.05	35.80	30.36	53.00	115.70
Gals. per capita	216	358	278	174	360
Per cent of max. 24 hrs.	158%	205%	222%	172%	165%
Storage to Equalize Hour Peaks					
In gals. per capita	24	33	31	14	50
In % of max. 24 hrs.	18%	19%	25%	14%	23%
Reduced Pumpage Peak by Storage in—					
Gals. per capita	80	184	154	73	140
Per cent of max. 24 hrs.	37%	51%	55%	42%	39%

How Storage Improves Pressure

Fig. 2 is a cross section showing ground occupied by a city, and certain lines illustrating available water pressure created by a pumping station.

The line AB illustrates the drop in pressure in a case where no elevated storage exists, and pump pressure is so used up in deficient pipes that no pressure remains at the peak hour of the heaviest pumpage (demand) day at the remote end of the town.

With an elevated tank installed the pressure conditions are changed as follows upon a similar heavy pumpage day. Assume the storage is such that the pumps instead of running at a variable rate run a constant rate day and night. During the peak hour of use pressure will be represented by the line AC representing water flowing from the pumps. The line DC represents water flowing from the tank toward the pumping station, and line DE,

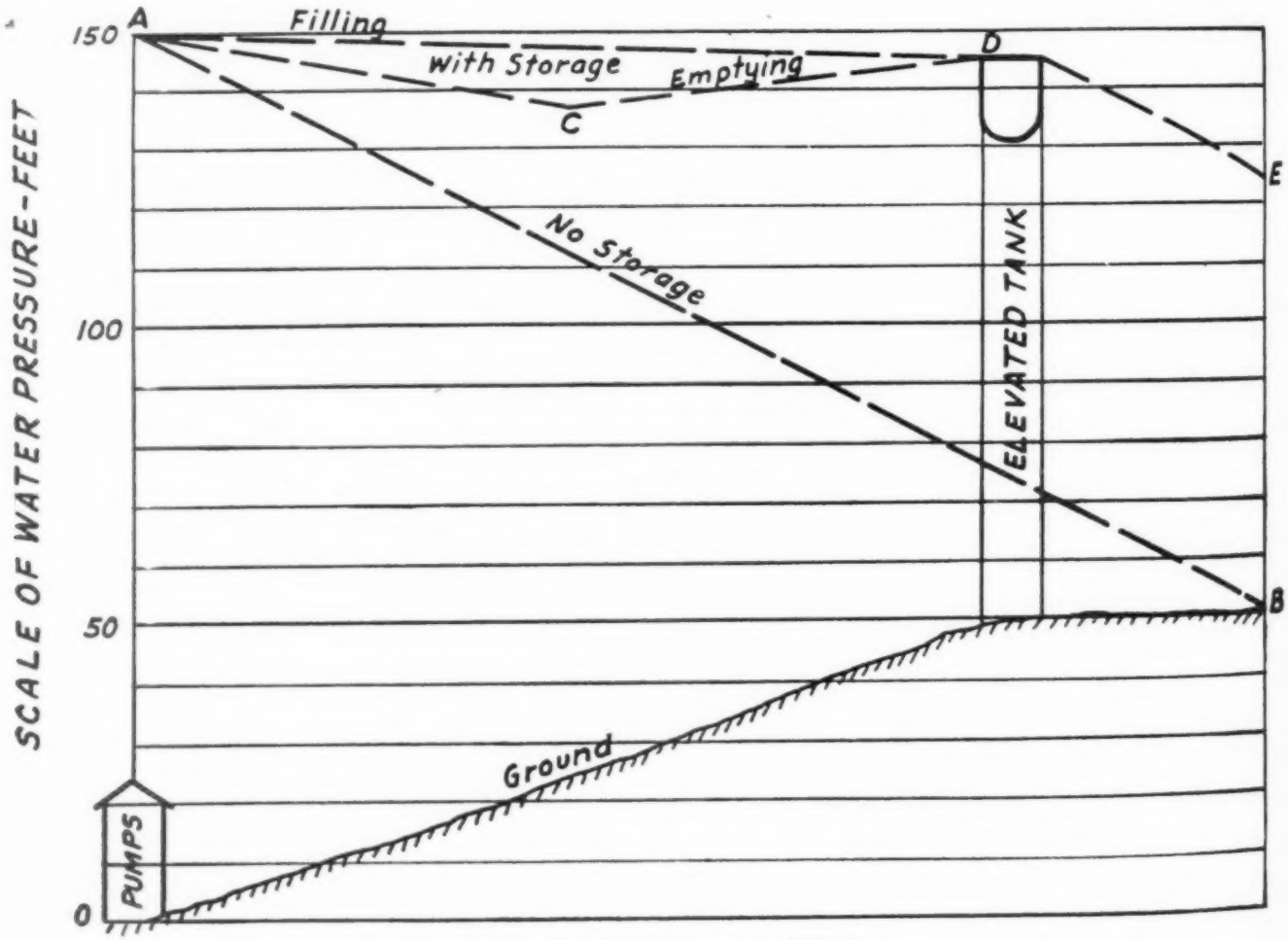

Fig. 2.—Effects of Distribution Storage Available Pressures
(Shown for a maximum demand day.)

flowing from the tank away from the pumping station. The pressure losses at points between tank and pumping station are smaller than before because this district is being fed from two ends. This results in shorter travel, smaller velocities and greatly reduced friction losses. At points beyond the tank friction losses are the same as before but pressure is increased because the water is coming from the tank at a high elevation.

The conditions noted prevail approximately during the peak hours when use exceeds pumpage. After the peak is past and pumpage exceeds use, during night hours, the pressure approaches the line AD which represents conditions just as the tank is filled. The lines AC, DC and DE will be lower as the tank is depleted; and when an empty tank begins to fill the line AD will be much steeper.

The above illustrates the general result when storage is added without increasing the distribution feeders. Storage is an alternative to new and larger pipes and pumps.

Location and Height

A number of local conditions enter into the question of the best location for storage and the most suitable elevation. Storage is almost always an addition to an existing plant and therefore it must fit existing facilities as well as possible. The following general principles should be guides:

1—Storage should be reasonably close to heavy peak drafts of water and on the opposite side of such points from the pumping station.

2—Elevated storage at the pumps helps the pumps but does not add to the capacity of the distribution system, which involves more than half the cost of the average water works.

3—Elevated storage on the opposite side of heavy draft permits water to be fed from both ends of the distribution system, thus nearly or quite doubling the capacity of the feeders.

4—The top of the stored water will necessarily be slightly below the maximum existing pressure plane of the pumps; but must *not* be too high to fill in the night hours of a peak day. The depth should be as shallow as reasonably possible; 20 to 25 feet is common.

5—The value of storage is largely lost when too far away. Costly pipes can overbalance the advantage of storage.

Amount of Storage Required

Fig. 3 shows the amount of storage necessary upon a maximum day to reduce the pumpage to a constant rate throughout the 24 hours of such a day. It also shows the effect of various lesser amounts of storage in reducing the peak hour requirement. The curve shown on the diagram is a median line between the extremes represented by dots upon the diagram. Each dot represents a computation for one of the five cities shown in Table I. It will be observed that a large benefit in pumpage reduction can be secured from a relatively small installation in storage.

These data indicate upon the average that 20 per cent of the maximum day in elevated storage may be expected to reduce the maximum hourly peak about 45 per cent. Or, expressed in another way, in the five cities studied 30 gallons per capita in storage might be expected to reduce the maximum hourly pumpage peak rate 126 gallons per capita if the storage is most effectively used.

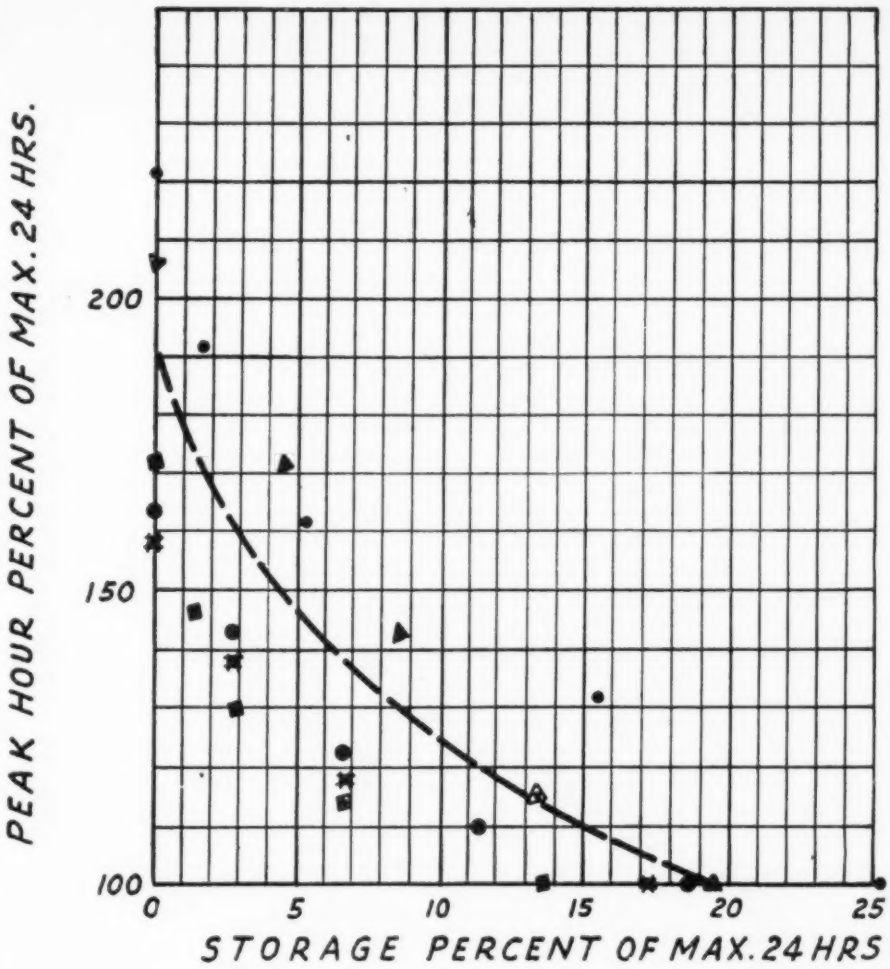

Fig. 3.—Effects of Storage on Pumpage Peaks in Five Cities
(Expressed in per cent of max. 24 hr. pumpage on maximum demand days. Each symbol of different design represents one of the five cities covered in Table I.)

In determining how much elevated storage shall be built, consideration is properly given to the existing installations in water supply facilities. The system as a whole must be designed for the maximum requirements for water, which must include the maximum requirements for fire protection if good fire protection service is furnished. In each case a proper installation is a coordination between elevated storage, water supply, pumps and distribution pipe facilities, so adjusting these facilities means that adequate service is furnished at least cost.

Fire Protection

One of the most important benefits from elevated storage is the ability to respond instantly to the heavy draft of water required for fire protection. In the smaller towns this furnishes an especially valuable service while pumps are being started, for many such systems pump intermittently.

In Table 2, column 4, there is shown the rates at which water should be furnished to comply with recommendations of the National Board of Fire Underwriters for cities of various sizes. For comparison there is also shown the average and maximum rates of pumpage based upon the average of the five cities shown in Table 1.

For the smallest town the fire demand is nearly five times the peak hour use, and pumps and distribution system must be designed accordingly without storage, bearing in mind that about 50 per cent above the average domestic demand plus the fire demand must be accommodated.

In towns over 25,000 the peak hour and the fire demand are about equal. In cities of 100,000 or more the combined domestic and fire demand is less than the domestic peak hour demand, and fire protection ceases to be a controlling factor so far as rates of pumpage are concerned.

It should be borne in mind that adequate fire protection provides for very rare emergencies. Likewise, the maximum storage for domestic peak hours is needed on only a few days in exceptional years, and usually an installation contains a reserve for future growth. Therefore an installation proportioned for domestic service is an important addition to the fire protection. It alone provides ample water for the great majority of fires, and the first and more important water in exceptional fires. Its great value is its readiness to serve.

Types of Distribution Storage

Various types of distribution storage are best adapted to special conditions. These types are summarized as follows:

1—Most of the cities in this country are relatively flat and the elevated tank of steel is most widely used. It is now built in shallow depths so that all the water is stored at sufficient elevation to be useful. Sizes as large as 2 mg. are available. Recent designs present a very pleasing appearance.

TABLE 2
Rates of Pumpage Required in Cities of Various Sizes

Population	1 Ave. MGD. at 100 Gals. per Capita 100%	2 Probable Peak 24-Hrs. MGD. 167%	3 Probable Peak Hr. MGD. 308%	4 Max. Rate for Fire Protection MGD.	5 Storage for Hourly Peaks 20% of Max. Day
1,000	.1	.17	.31	1.44	.033
2,000	.2	.33	.61	2.15	.066
4,000	.4	.67	1.24	2.86	.134
10,000	1.0	1.67	3.08	4.30	.334
28,000	2.8	4.66	8.64	7.15	.932
60,000	6.0	10.00	18.50	10.00	2.00
100,000	10.0	16.70	30.80	12.90	3.34
200,000	20.0	33.40	61.60	17.20	6.68

2—There are some situations where elevated ground is available but not high enough for a concrete reservoir on the ground. In such situations circular steel tanks up to 40 to 60 ft. high and of large capacity can be built for less than half the cost of elevated tanks per unit of capacity. Such tanks can sometimes be scattered at more than one location with beneficial results. For the higher of such tanks all the water may not be fully useful but it furnishes a cheap support for the more useful elevated water.

3—Many cities are so fortunate as to have elevated ground sufficiently high to accommodate elevated reservoirs. These cost about half as much as most elevated tanks. Most recent reservoirs are covered, built of reinforced concrete buried in the ground, and have long life with small maintenance costs. Reasonable cost favors large capacities. In some installations the entire fire protection water is stored in addition to domestic requirements. As much as 100 to 300 gallons per capita is stored in some cities.

4—Where elevated ground is not available it is sometimes economical to build concrete reservoirs or steel tanks on the ground, fill them at night with water bled from the mains and repump stored water back into the mains by remote controlled or automatically controlled electric pumps. The capitalized cost of repumping should, of course, be charged to the installation. In cost comparisons such pumping may be less costly than might be thought if operation is required only a few hours per day during a limited portion of the year.

Operation of Storage

Money should be conserved for use. Likewise water stored should be conserved so that it will be available when needed. There is little use for storage during about 80 per cent of the normal year. With storage it becomes possible to run pumps at a nearly uniform rate, with attendant advantages of economy, and on normal days to utilize only a few feet of the upper part of the storage. The storage should only be materially exhausted during the peak hours of the heavy pumpage days, and always refilled before morning.

Water works men are now more firm than ever in the belief that storage on the distribution system pays its way in the reliability and economy of water service.

FIRE HYDRANT FLOW GAGE

By A. A. HIRSCH*
State Dept. of Education, Baton Rouge, La.

Of the several methods in vogue for hydrant flow measurement, namely the special pitot tube-gage combination, the stream trajectory, the short tube with terminal orifice, and the velocity head reading by means of a pressure gage on a closed cap, the last named is the one most generally employed by reason of its simplicity in principal and equipment.

The Procedure

In determining flow from a fire hydrant by the pressure gage method all that is needed is to connect an accurate gage to the closed nozzle, and discharge the hydrant through the other nozzle. The flowing nozzle may be considered as a "short-tube"

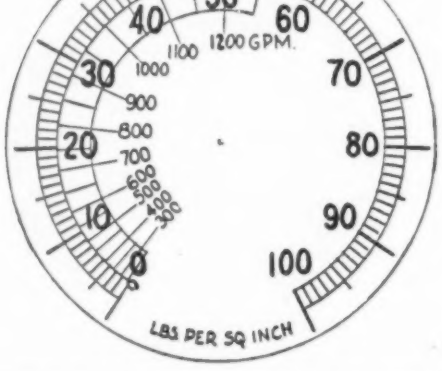

Figure 1.—Dial of pressure gage calibrated to read directly fire hydrant flow from a 2½-inch nozzle.

and its flow calculated from the usual orifice formula, but using an orifice coefficient of 0.9 instead of the familiar 0.61 which holds for a sharp edged orifice. Thus

Flow (cu. ft./sec.) = C A V where C = fire hydrant orifice coefficient = 0.9.

$$A = \text{area of nozzle (sq. ft.)} = 0.785 \times \frac{\text{Diameter}^2_{(\text{inches})}}{144} = 0.00545\ \text{Diameter}^2_{(\text{inches})}$$

$$V = \sqrt{2\,g\,h_{\text{ft. water}}} = \sqrt{2 \times 32.16 \times 2.31 \times \text{Pressure}_{\text{psi.}}}$$ the pressure reading being taken off the closed nozzle

$$= 12.2\sqrt{\text{Pressure}_{\text{psi.}}}$$

Substituting in the first equation:

$$\text{Flow (gpm.)} = 7.48 \times 60 \times 0.9 \times 0.00545\ \text{Diameter}^2 \times 12.2\sqrt{\text{Pressure}_{\text{psi.}}}$$

$$= 27\ \text{Diameter}^2_{(\text{inches})}\sqrt{\text{Pressure}_{\text{psi.}}}$$

This is the familiar form commonly used to calculate the discharge from a fire hydrant.

Ordinarily the 2½-inch nozzle is used, as the 4- or 4½-inch nozzles do not conform with accuracy to a simple formula. Substituting the value of 2½ inches for the diameter in the above formula there results

$$\text{Flow (gpm. through 2½ in. nozzle)} = 27 \times 2.5^2 \times \sqrt{\text{Pressure}_{\text{psi.}}} = 168.7\sqrt{\text{Pressure}_{\text{psi.}}}$$

from which the discharge through a 2½-inch hydrant nozzle may be conveniently calculated.

The Self-Calculating Gage

Inasmuch as a given velocity head on the pressure gage corresponds to a fixed discharge the gage may be calibrated to read this flow directly and thereby obviate the need for calculation. A dial so calibrated is illustrated in Fig. 1. By marking off a dial in this manner the gage may be used in hydrant flow studies to facilitate discharge adjustments and to enable flow evaluation on the spot without calculations or conversions. Fig. 1 may also be used as an approximate solution to the hydrant formula for a 2½-inch nozzle rather than perform the numerical substitutions.

The accuracy of the pressure gage should be checked on a cold test apparatus, especially in the low pressure range below 500 gpm. where slight inaccuracy causes relatively large errors in the indicated discharge. If the test gage is off the needle position may be reset, the linkage between hollow spring and sector adjusted as required, or a calibration curve drawn to show the true pressure corresponding to any gage reading. From Fig. 1 it will be noted that below a flow of about 250 gpm. the type of pressure gage as illustrated is inapplicable since it fails to deflect.

CATHODIC PROTECTION OF STEEL SURFACES IN CONTACT WITH WATER

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PART I—Basic Principles and Controlling Phenomena

THE establishment of the most satisfactory method for mitigating the corrosion of steel surfaces in contact with natural waters[1] in the pH range of 4.5 to 10.0 has been a challenge to all engineers who design and operate steel structures such as clarifiers, filters, water storage and heating tanks which come in contact with this environment. The method selected for retarding the wastage of metal by corrosion is usually based on any one or a combination of the following procedures: (1) use of protective coating; (2) the adaptation of metals and alloys which resist the corrosion environment encountered; (3) chemical treatment of the water to make it less aggressive; (4) cathodic protection, which subdues galvanic corrosion electrically.

In the water works field, cathodic protection as a method of corrosion control is a relative newcomer, the first application to water storage tanks probably being made in 1936.* Since that time cathodic protection has established itself, when properly applied, as being a practical method of protecting submerged steel surfaces.

What Cathodic Protection Is

In the simplest sense, cathodic protection consists of stifling the flow of current which accompanies electrochemical corrosion by projecting a current flow through the corroding media to the metal where it becomes superimposed on the metal surface to be protected.

This definition conforms with that given by earlier research workers in the field of corrosion, and its thought will be developed throughout this paper.

When compared with other methods of corrosion control, properly understood and properly applied, cathodic protection has advantages which justify its consideration as the solution of many corrosion problems.

(a) In many cases installations can be made and placed in operation without taking the equipment to be protected out of service. This is of particular importance in water tanks used for fire protection, water systems where there is only one head tank or in other equipment where an outage would interfere with production.

(b) Surfaces to be protected do not require special preparation. Old paint films and most scale and corrosion products actually reduce current density requirements for complete protection.

This eliminates the necessity of sand blasting, flame cleaning or wire brushing to remove the old coating and corrosion debris, drying of the exposed surface and providing adequate setting time for a protective coating before contact with the water, in ordor to obtain a satisfactory protection for underwater surfaces.

(c) The use of an adequate current density to take care of the cathodic processes on the surfaces exposed to the corroding media will give complete protection by maintaining protecting effect continuously on the metal surface. This may be compared with the development of failure in most protective coatings which become focal points of accelerated corrosion, resulting in deep pits before a new covering can be applied.

(d) Cathodic protection requires very little maintenance and supervisory attention.

(e) Cathodic protection can be economically justified when compared with other methods of underwater corrosion control.

(f) The chemical nature of the bulk of the water is not altered by cathodic protection.

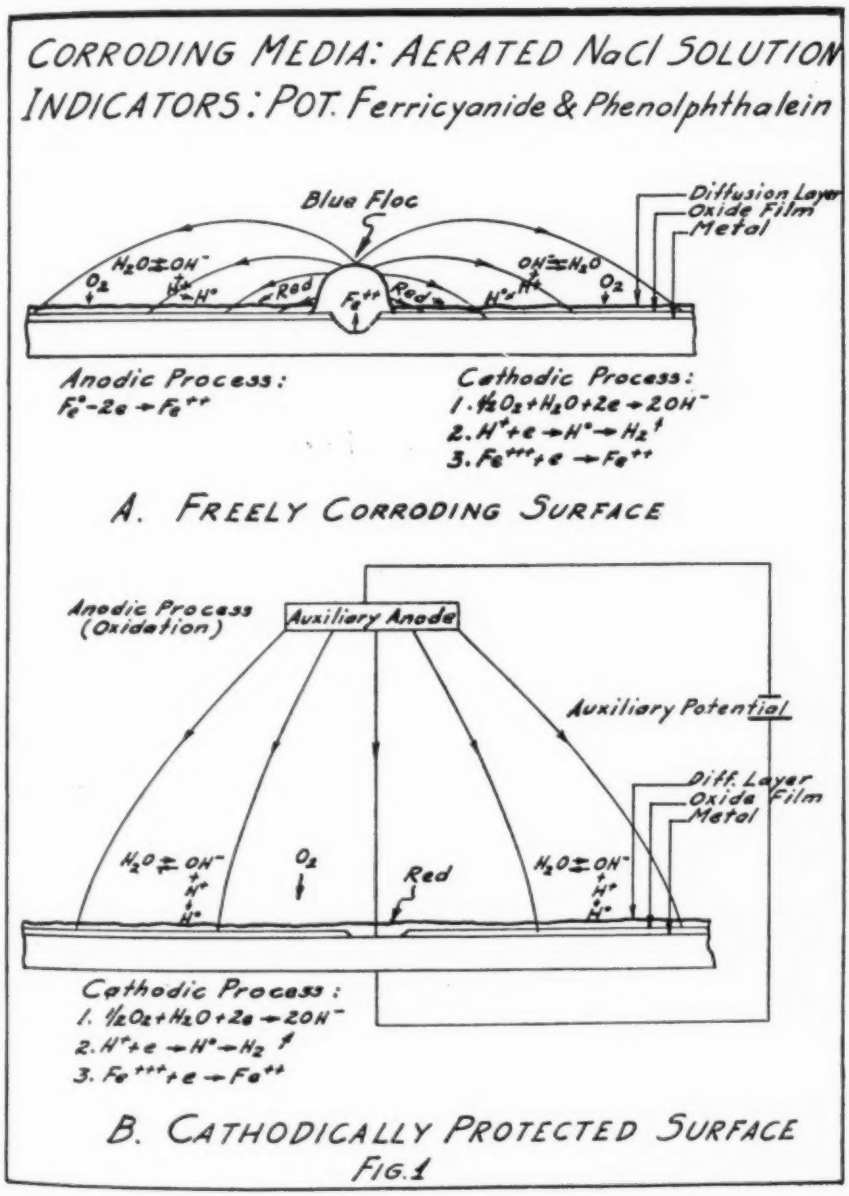

Fig. 1 (A & B)—Schematic Illustration of Chemical Reactions at the Face of a Freely Corroding Surface and One Which Is Receiving Cathodic Protection.

*"A New Method of Tank Protection"—O. B. Hess, Supt. of Water, St. Clairsville, Ohio (*W. W. & S.*, Nov., 1938) is seemingly the earliest published report on cathodic protection applied to water tanks.

Background

The idea of cathodic protection is not all new, since Sir Humphrey Davy[3] in 1824 observed that a metal "...if it could be rendered slightly negative, the corroding action of sea water upon it would be null..." and he "...thought of using voltaic battery..." to supply the current flow to the metal surface.

Practical limitations of the voltaic cells in those days prevented Davy from using them as a means of motivating current flow to large surfaces to be protected; however, he did attach zinc plates to these metal surfaces and protected them by the flow of current generated by the potential difference in the zinc-metal couple. The zinc plates were not effective over large areas. The most complete protection occurred adjacent to the zinc and rapidly diminished in effectiveness as the distance from the zinc was increased.

Around 1912 Cumberland[3] in England, and in the early 1920's Kirkaldy[4] in the United States, made use of direct current for corrosion control and the prevention of scale and formation in surface condensers and boilers. In his application of cathodic protection Cumberland installed anodes of iron mounted on insulated supports in the water boxes of the condenser. By means of direct current from a dynamo he caused current flow from the iron anodes through the sea water to the surfaces to be protected.

The Kirkaldy system was similar to the Cumberland system and was based on the idea that any metal which was made sufficiently cathodic to eliminate all local voltaic or galvanic couples would neither corrode nor permit scale to develop.

Within the last two decades cathodic protection[5] has been used for underground pipeline corrosion control and has probably attained its greatest commercial application in this field, particularly in the petroleum and natural gas industries.

Although the art of cathodic protection is quite old, its practical application has until recent years outdistanced the fundamental engineering data available for design. In order to properly design a cathodic protection installation it is necessary to thoroughly understand the corrosion mechanism itself, the factors in the corrosion environment which govern current density requirements, the fundamentals of electrode materials and design, the economic considerations involved in the initial and operating costs, sources of potential for motivating current flow to the submerged metal surfaces and hazards of the improper use of electric current. These are only a few of the considerations involved in the engineering design of a cathodic protection installation for a particular corrosion problem and indicate that this work should be done only by engineers experienced in the application of cathodic protection.

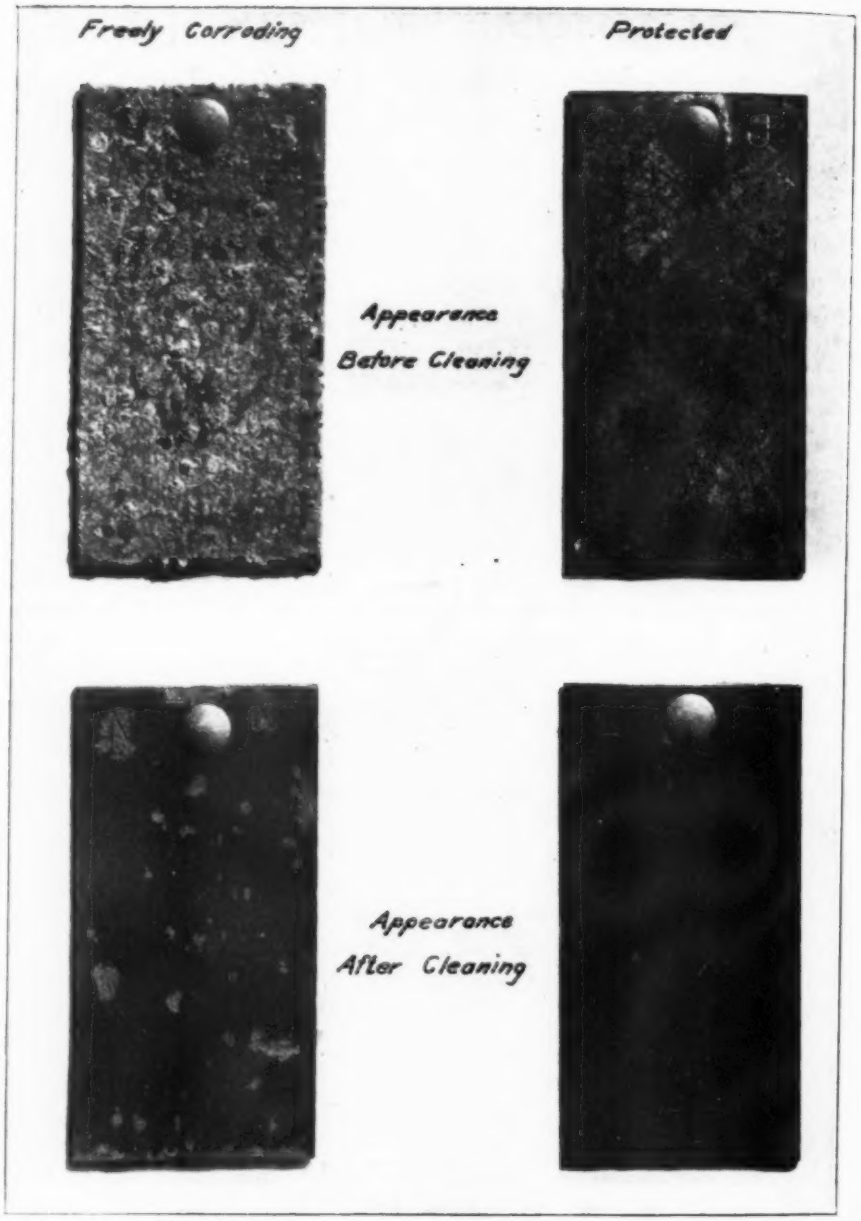

Fig. 2—Freely Corroding and Cathodically Protected Test Specimens from a Dorr Clarifier. (Left—Freely corroding pieces before and after cleaning; right—protected pieces before and after cleaning. Note chalky deposit on plate J before cleaning. Note pitmarks on plate N after corrosion products seen on plate G had been removed.)

Corrosion Mechanism

In order to visualize the manner in which a current flow through the corroding media (water) to a steel surface will protect the metal, the mechanism of electrochemical corrosion should be considered and particularly the factors governing the rate of corrosion.

The localized type of attack illustrated by the development of pits on freely corroding steel surfaces exposed to aerated natural waters (Fig. 2) has been shown to be related to the flow of current resulting from differences in the tendency for the metal to enter the solution in contact with it. Non-uniformities of the metal surface or of the solution in contact with the metal can cause these differences in potential. In practice (1) differential thermal treatment, (2) localized failures of the protective oxide film, (3) differential concentration or nature of the corroding media, (4) differential aeration and (5) dissimilar metals generally determine the location and intensity of the corrosion current.[6]

The electrochemical theory of corrosion proposed in 1903 by W. R. Whitney[7] has become the basic concept of most corrosion phenomena in electrolytes. He associated corrosion with the flow of electricity resulting from the loss of two electrons by each iron atom leaving the crystal lattice of the metal and entering the solution as a ferrous ion at the anodic or most electronegative areas on the metal surface.

$$[Fe^{\circ} - 2e \rightarrow Fe^{++}]$$

This equation pictures the solid metal going into the soluble state involving ionization.

At the cathodic or less electronegative areas, the electrons (e) released by this process are accepted by an equivalent number of hydrogen ions at the metal-solution inter-face where they are reduced to hydrogen atoms. By reducing the hydrogen ions to hydrogen atoms at the more cathodic areas the hydroxyl ions, which result from the dissociation of water, concentrate there and increase the pH.

$$(1)\quad 2H_2O \rightleftarrows 2H^+ + 2OH^-$$
$$(2)\quad 2H^+ + 2e \rightarrow 2H^{\circ}$$

This process if undisturbed develops sufficient counter potential (polarization) on the cathodic areas in airfree neutral waters,

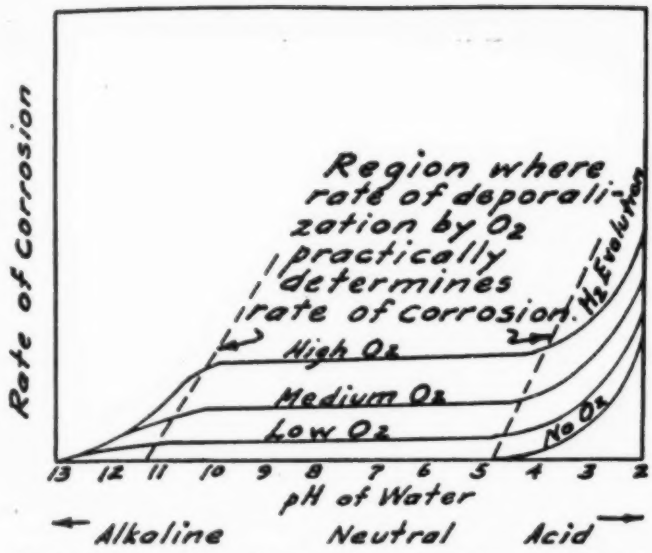

Fig. 3—Effect of pH on the Corrosion Rate of Iron at High and Low Dissolved Oxygen Content of the Water.—(Wilson, 1923).

to stifle further deposition of hydrogen ions and in turn retard solution of metal at the anodic areas.

This mechanism can be illustrated by observing the surface of a steel plate (Fig. 1A) submerged in a quiescent distilled water solution containing sodium chloride and a modified "ferroxyl indicator" solution. The "ferroxyl indicator" solution consists of potassium ferricyanide and phenolphthalein.

The *anodic* process is shown by the formation of a blue floc (Prussian blue) resulting from the reaction of the ferrous ion and the potassium ferricyanide. This blue floc is essentially the corrosion product (rust) forming when the pitting action is taking place. The amount of tuberculation or rust, which is much more voluminous than the metal from which it was formed, is equivalent to the iron atoms which have left the metal structure and entered the corroding media in the form of ferrous iron (Fe^{++}).

The *cathodic* processes occurring over most of the metal surface are shown by the development of red color with the phenolphthalein indicator. No blue floc is found on these areas, indicating that there is no loss of metal at such points.

Thus the freely corroding metal surface in contact with the corroding media may be pictured as consisting of many irregular shaped and spottily distributed areas at which metal wastage is occurring, while the remainder of the surface is free of attack. The flow of current from the local anodic areas through the corroding media is essentially supplying a degree of natural cathodic protection to the rest of the exposed surface. The magnitude of the corrosion current and in turn an equivalent amount of metal wastage at the local anodic areas, is governed by the rate of depolarization occurring on the cathodic surfaces.

Unfortunately in the case of iron and most other metals, there are processes and acceptors (depolarizers) for the electrons released by the anodic process which do not permit the more cathodic areas to polarize to the open circuit potential of the anodic areas. Hadley[9] has enumerated five cathodic processes which may govern the rate of electrochemical corrosion. The first three are well known and may be observed to varying degrees in natural waters.

1. Combination with dissolved oxygen.

$$[H\text{-}OH + \tfrac{1}{2}O_2 + 2e \rightarrow 2OH^-]$$

2. Depolarization by agitation or bubbling off of hydrogen.

$$[2H^+ + 2e \rightarrow 2H^\circ \rightarrow H_2]$$

3. Direct chemical combination with the electrolyte or salts, for example the reduction of ferric ions to ferrous ions.

$$[2Fe^{+++} + 2e \rightarrow 2Fe^{++}]$$

4. By entering into the metabolic processes of certain anaerobic bacteria.

5. Combination with the products of microbiological metabolic processes.

The last two methods mainly occur in airfree soils and waters. Fig. 3 was developed by Wilson[1] in 1923 to indicate the relative influence on the corrosion rate of iron by the pH and dissolved oxygen concentration of the water.

From this it is evident that the corrosion rate in natural waters is governed mainly by the dissolved oxygen depolarization process, although it is known that the hydrogen evolution process occurs simultaneously.

For all practical purposes the rate of corrosion represented by hydrogen evolution is not significant in the absence of dissolved oxygen until the pH is less than 5.0 at normal temperatures.

When the acidity of the electrolyte drops below pH 5.0, the hydrogen ion pressure is sufficient to overcome the hydrogen overvoltage at the metal surface and cause marked evolution of hydrogen gas. This is the acid type of attack.

To further illustrate the key part that dissolved oxygen (O_2) plays in the rate of corrosion in neutral waters,[10] it has been noted that with all other conditions equal the corrosion rate of iron increases linearly with the dissolved oxygen concentration up to 5.5 cc. per liter.

Current Density Requirements

Although an excessive current density will not prove to be harmful to iron as it does to the more amphoteric metals such as aluminum,[11] the cost of the original installation and of operation dictate that the minimum current density requirements be established for complete protection against the corrosion environment being encountered.

Current density requirements for any corrosion problem have been generally chosen on a "rule of thumb" basis, which the worker may feel is adequate for a specific type of problem. Davy[3] indirectly established the minimum current density requirements for the protection of copper against sea water by observing that the ratio of copper to zinc must be less than 150 to 1 to obtain complete protection.

An examination of the literature on cathodic protection indicated current density requirements ranging from 0.3 to 60 mille amperes per sq. ft. for the protection of iron against various water solutions. Generally very little thought has been given to the factors which may cause this wide variation in current density requirements.

Clement and Walter[12] in some of the first fundamental work done on cathodic protection established two conditions affecting the c.d. requirements for the protection of steel against O.01N H_2SO_4.

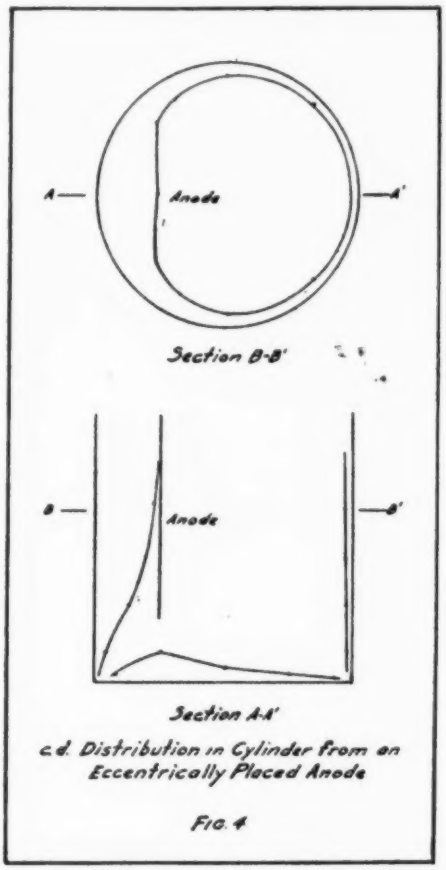

Fig. 4—Results of a Current Density Distribution Study Revealing Facts Contrary to Expectancy.

First, they observed that the current density requirements increase with increasing dissolved oxygen concentration.

Secondly, with all other conditions being equal, increasing the stirring increases the current density requirements.

These observations may be visualized more readily if we consider the presence of a stagnant layer of solution existing at the metal surface. This stagnant layer (Fig. 1A and B) acts as a barrier to the diffusion of dissolved oxygen from the bulk of the water to the metal surface. If we increase the dissolved oxygen concentration in the water we increase the number of oxygen molecules reaching the metal surface. By increasing the temperature or the velocity of the corroding media next to the metal surface we reduce the effective thickness of the diffusion layer and increase the rate of oxygen diffusion.

Since the rate at which dissolved oxygen reaches the cathodic areas through the diffusion layer governs the rate of metal wastage in natural waters, we can see that the superimposed current from an auxiliary anode (Fig. 1B) required to retard the rate of corrosion at the natural anodes will also be influenced by the dissolved oxygen concentration and the factors controlling the effective thickness of the diffusion layer.

Work done by the writer has indicated that in quasi-stagnant corrosion media of water containing less than 1.0 ppm. of dissolved solids, 0.1 N and 1 N NaCl solutions, with controlled oxygen concentration in the bulk of the media, the current density requirements for complete protection vary directly with the dissolved oxygen concentration and is relatively independent of the salt concentration.

The Effects of Calcium and Magnesium Salts

The degree of lime saturation of the natural water markedly affects the current density requirements for metal protection. A high degree of lime saturation in a water, as shown by the Langelier Saturation Index, inhibits the corrosion process. This is true because the smallest natural corrosion current develops sufficient alkalinity on the cathodic areas to precipitate a deposit of $CaCO_3$ from this water, increasing the effective thickness of the diffusion barrier and in turn reducing the amount of oxygen reaching the cathodically active points on the submerged metal surface. However, the saturation of a water with calcium carbonate and a pH of 10.3, such as that found in the water in contact with the freely corroding specimen (Fig. 2) should not be relied upon alone to completely stifle the corrosion process in the presence of appreciable amounts of dissolved oxygen.

The ordered reduction of the hydrogen ions and the migration of the cations such as Ca^{++}, Mg^{++} and Na^{+} to the protected cathode surface when projected current is applied, produces a highly alkaline film at the metal surface which is indicated by the red color developing when the "ferroxyl indicator" is used (Fig. 1B). The calcium bicarbonate $Ca(HCO_3)_2$ and/or magnesium ions in the bulk of the water coming in contact with the highly alkaline solution developed at the boundary of the diffusion layer precipitates calcium carbonate ($CaCO_3$) and/or $Mg(OH)_2$ at the metal surface. This often appears as a chalky coating on a protected metal surface. [Note uncleaned protected specimen, Fig. 2.]

Even waters with very little $Ca(HCO_3)_2$ and magnesium ion will deposit a $CaCo_3$ and/or $Mg.(OH)_2$ film providing the current density being applied produces a sufficiently high alkalinity at the metal surface.

From the foregoing it should not be assumed that cathodic protection is *entirely* dependent on the presence of calcium and magnesium salts in the water for effectiveness. The writer has protected steel surfaces immersed in aerated water containing less than 1.0 ppm. of dissolved solids. Clement and Walker[13] and U. R. Evans[14] have protected steel cathodically against dilute acid solutions. The advantage of calcium and magnesium salts in water is that the current density requirements for complete protection are lower in their presence and consequently make for greater economy of process.

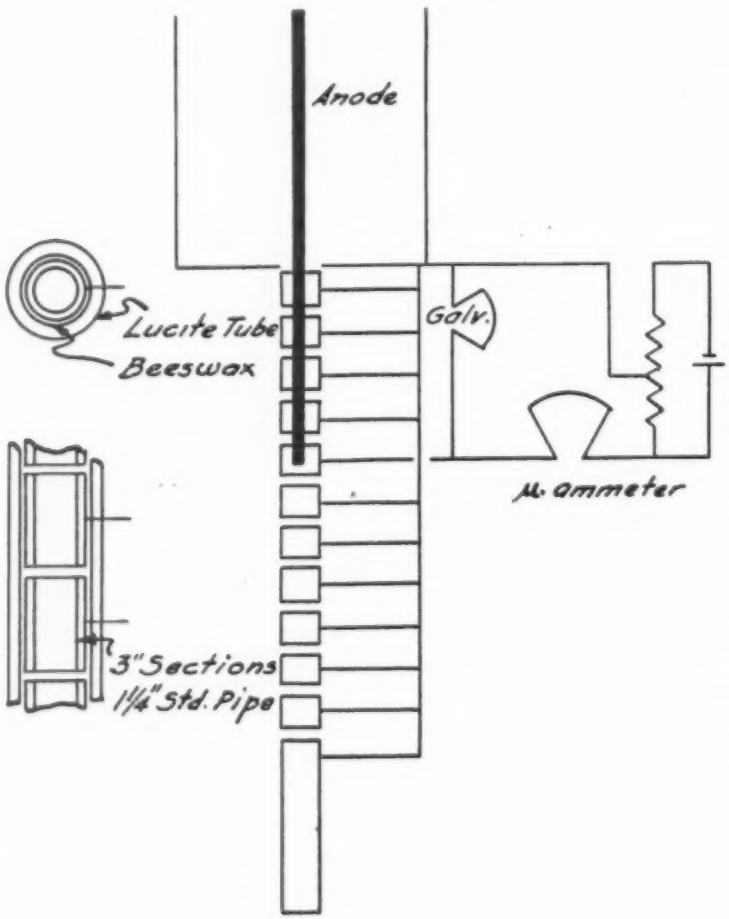

Fig. 5—Experimental Arrangement in a Study of Current Density Distribution in Tubes and Elevated Tank Risers.

The Effect of Protective Coatings

Paint and asphaltic coatings, even though they may be in a very poor condition, will reduce current density requirements since they are effective diffusion barriers and by the electrical resistance of their films concentrate the flow of current to points of film failure where the metal is exposed. Experience in the application of cathodic protection to underground pipelines has shown that the condition of the protective coating markedly influences the current density requirements.

Since the natural corrosion current is governed by the depolarization rates occurring on the more cathodic areas of the steel, the projection of an applied current from a submerged auxiliary anode to the metal surface will reduce the current leaving the local anodic areas on the steel. Increasing the applied current reduces the local current flow until the "protective current density" is reached, at which point all local corrosion currents have been stifled. It has been observed in 1N salt solutions that the potential of the steel does not change until the "protective current density" is reached, since the freely corroding steel surface potential is governed by that of the cathodic areas and the actual current density flowing to the cathodic areas has not been altered up to this point. Instead of supplying the current from the local anodes on the metal surface it is now supplied from the auxiliary anode in the corroding media (Fig. 1B). Above the "protective current density" the potential[14] of the specimen behaves similarly to a hydrogen electrode. Ewing has shown that a measurement of the metal potential alone is not a criteria of protection.

M. DeKay Thompson[9] has observed that the degree of protection of steel surfaces in NaCl and Na_2SO_4 solutions by an applied current is directly proportional to the cur-

rent density applied up to the "protective current density." This suggests that the use of insufficient amounts of applied current for *complete* protection are still of benefit since it will tend to reduce the local corrosion currents in any event. He also observed that in the same environment the current densities required to protect cast iron as against steels did not differ markedly.

It should not be assumed from the foregoing that the required current density for protection is just equivalent to the natural corrosion current represented by the metal wastage. There are many factors which do not permit this assumption to be valid. One of these may be visualized if we consider the cathodic areas in natural corrosion to represent only a portion of the exposed surface. When an applied current is used some of it flows to the areas which were anodes in normal corrosion. Often it is the case that many times the natural corrosion current must be applied from the auxiliary anodes to completely stifle the corrosion process.

Anode Design

The criteria of good anode design is the proper distribution of the current density, long anode life and the establishment of a low resistance through the corroding media between the anode configuration and the surfaces to be protected. It may be thought that the centering of an anode in a water tank with a flat bottom would provide a uniform current density to all surfaces which are equidistant from the anode. In the slightly more complex case of an eccentrically placed anode in a steel cylinder with a conducting floor (see Fig. 4) the current density distribution determined experimentally by the writer does not substantiate this thought. [The current density distribution was obtained by a probe technique developed by the writer.* Also, this probe technique has been employed to study the current distribution over the surfaces of complex structures from an operating cathodic protection installation, such as a Dorr clarifier mechanism.]

In order to picture the distribution of current in condenser and boiler tubes, pipes or in elevated tank risers, a test assembly (Fig. 5) was constructed of sections of pipe insulated from one another. Conductors led from each section were grounded to the tank which represented the water box of a condenser, drum of a boiler or bowl of an elevated tank. The current density on any section could be determined by dividing the current measured flowing to the section in question, by the surface area of the section. Table I illustrates the current density flowing to each section from a centered 3/16-in. diameter electrode extending to the bottom of the fifth section using a 5 per cent NaCl as the electrolyte. (See Fig. 5.)

TABLE I

Section	Current Density (Average)
1	4.3 micro amps/sq. cm.
2	4.0 "
3	4.5 "
4	4.0 "
5*	4.0 "
6	0.5 "
7	0.2 "
8	0.0 "

*Anode terminated in Section 5.

These data indicate that the current distribution extends only a short distance from the end of an electrode within a cylinder. From this it may be concluded that the anode design should be made to properly distribute current without any reliance on the so-called "throwing power" of the current input.

The ideal anode material would be one which resists deterioration when current flows from its surface, is inexpensive, does not introduce deleterious products into the electrolyte, is mechanically strong, has a low resistance and may be fabricated in sizes and shapes required to conform to the conditions encountered. There is no perfect anode material for all problems and the conditions found in the application will dictate the material used. Amongst the materials generally considered are carbon, graphite, mild steel, chrome and stainless steels, duriron, aluminum, zincs, copper, platinum, gold, etc.

The theoretical volumes and weights which will be lost by metals per ampere year of current flow are revealed in Table II. In the case of graphite the mechanical loss due to sloughing off of the particles of graphite are often much greater than the electro-chemical values would indicate.

TABLE II

Electrochemical Equivalents of Anodes

Anode	Sp. Grav.	Lbs./Amp. Yr.	Cu. in./Amp. Yr.
Iron	7.86	20.1	71.0
Graphite	2.26	2.2	27.0
Zinc	7.14	23.5	91.4
Nickel	8.90	21.3	66.5
Aluminum	2.702	6.4	65.7

*Reported on in a paper presented by the author to the Fifth Annual Water Conf. of the Engineers' Soc. of Western Pennsylvania, October, 1944.—Ed.

Water Quality and Current Density

A paradox found in the application of cathodic protection is the observation that for waters of high resistance (low solids content), higher current densities are usually required than for waters of low resistance. The implication of this thought may be gathered when we compare midwestern waters with a specific resistance of 1200 ohms per cm. cubed with that found along the eastern seaboard, whose specific resistance is often 20 times as great. For the same tank and electrode arrangement, 20 times the power consumption would be required to supply the same current to the eastern tank. A reduction in power consumption can be accomplished only by altering the "cell constant" of the electrode-tank configuration. This can be done by increasing the number and altering the spacing of the electrodes and/or increasing the ratio of the anode diameter to the tank diameter. For an anode centered in a cylinder,[13] the effect of the ratio of the anode to tank diameters upon the resistance may be predicted from the following relationship:

$$\text{Resist.} = \frac{2.303}{2\pi} r \log_{10} \left[\frac{R_2}{R_1}\right]$$

where

r sp. resistance
R_1 Anode diameter
R_2 Cylinder diameter

Part II—Comments on the Practical Application of the Process

IN ORDER to mitigate the corrosion process of steel surfaces in contact with natural waters by cathodic protection, it has been shown in Part I that for each corrosion environment a certain "minimum current density" must be projected from an auxiliary anode configuration through the water to the surface to be protected. The "minimum current density" required to stifle the local corrosion currents on steel surfaces in a natural water environment is largely governed by the coloumb-second or ampere-hour effect occurring in the electrochemical reduction (a) of the dissolved oxygen reaching a unit area of the metal surface through the diffusion layer existing between the metal surface and the bulk of the water.

(a) $HOH + \frac{1}{2}O_2 + 2e \rightarrow 2OH^-$

From these principles previously discussed it becomes apparent that the effectiveness of cathodic protection is independent of the source of the projected current so long as the amount is adequate.

Sources of Protective Current

The flow of protective current may be motivated in one of two manners:

(1) Using a metal plate whose solution potential in the particular corrosion environment is greater (more anodic) than that of the metal to be protected, a protective current can be produced. The potential difference between the two metals in the corroding media results in the flow of current from the more anodic or sacrificial metal through the solution to the metal surface being protected, if an electric circuit is provided between the metal being protected and the anode. An example of this method of corrosion control, which may be better described by the term "galvanic protection" rather than "cathodic protection" is the use of zinc plates coupled to steel surfaces. Metals such as cadmium, aluminum, and magnesium have also been found to have a protective effect when coupled with iron. Another example whereby the potential difference between two metals may be used to protect the less anodic metal was illustrated by Davy[11] when he used iron, which in most environments is anodic to copper, as a sacrificial anode to protect the copper sheathing of ships in sea water. [We are all familiar with the phenomenon occurring when iron is coupled with copper in aerated waters. The iron immediately adjacent to the copper is usually severely attacked, with the degree of attack on the iron diminishing as the distance from the copper is increased.]

There are two main limitations in the use of zinc plates or some of the other more anodic metals for providing the necessary "protective current density" on steel surfaces. Reviewing the electrochemical equivalents of metals in the first article of this series, it was shown that at least 23.5 pounds of relatively expensive zinc would be consumed per ampere year if there were no other anodic processes occurring. This zinc consumption, resulting from a current flow of one ampere for one year to the steel surface, does not include the zinc loss occurring because of the local corrosion currents flowing over the zinc surface. In

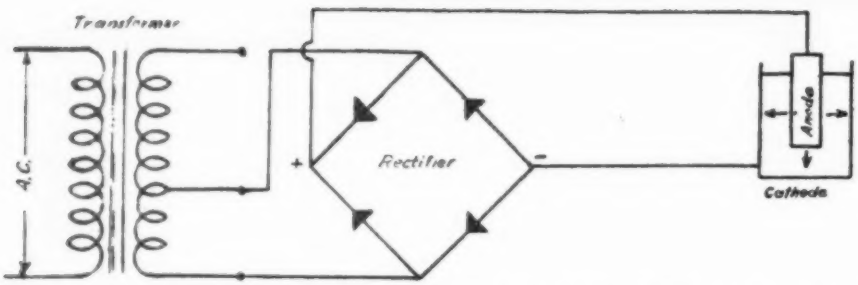

Fig. 6—Diagram of Simplified Cathodic Protection Circuit Utilizing a Dry Type Rectifier.

most cases experience has shown that the cost of zinc plates, which serve both as an anode and a potential force, is prohibitive.

The shortcoming of this galvanic method is that the potential difference existing between the more anodic or sacrificial metal and the metal to be protected usually is not sufficient to overcome the resistance through the corroding media and supply an adequate current density to completely stifle the corrosion currents over large areas. It has already been stated that the application of a current density below the "minimum protective current density" will give only partial protection. Because of the limited potential driving force available, complete protection usually extends only a few inches away from the zinc plates. Beyond the outer edge of the area, where complete protection is observed, the protective effect diminishes as the distance from the zinc increases because of the gradual decrease of the current density flowing to the surface of the metal being protected.

(2) The second and most used method of projecting current flow from a properly designed auxiliary electrode configuration in the corroding media to the surfaces to be protected is with an external potential force which can be regulated to drive the current required for protection.

The external potential force to be applied to the auxiliary anodes in the form of direct current can be developed by storage batteries, wet type rectifiers, dry type rectifiers, motor-generator sets, gas-engine generators and windmill generators.

Because of the conditions met in the practical application of cathodic protection, the equipment selected for this purpose should have the following characteristics: low initial cost, require very little maintenance, have a high efficiency in the conversion of energy to the proper direct current, be easily adjustable to meet potential and current requirements of the corrosion control installation, be unaffected by weather and climate, attractive in appearance, operate continuously, etc.

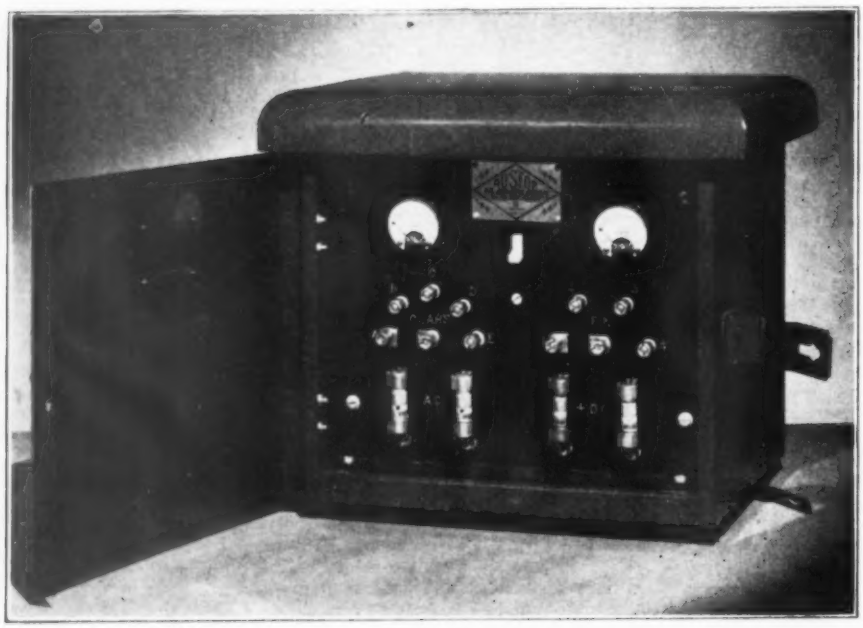

Fig. 7—Copper Oxide Rectifier Used in Cathodic Protection of Water Tanks and Sewage Plant Equipment.

Rectifiers

The equipment most frequently selected for developing the external potential force for driving the current in the application of cathodic protection to pipelines and submerged steel structures is the dry type rectifier. There are three well developed dry type rectifiers commercially available at the present time. These are generally referred to as copper oxide, selenium, and copper sulphide rectifiers. These rectifiers are dependent upon the electrical check-valve properties of certain metals for converting alternating current to direct current. It has been found that the formation of copper oxide on one side of a copper disc of high purity, by controlled exposure to heat and air, creates a unique electrical check-valve property. By making an electrical contact to the metallic copper, and another through lead discs in contact with the copper oxide formed on the surface of the copper, it is observed that unidirectional current will encounter less resistance and flow more readily from the copper oxide film to the copper than in the reverse direction; i.e., from copper to copper oxide. Basically, the selenium and copper sulphide rectifiers function on the same principle as the copper oxide rectifier does. By using this check-valve property, rectifier stacks can be assembled for converting alternating current to unidirectional or direct current, using the full wave of the a-c cycle. Fig. 6 is a simple diagram to show the position of the rectifier stack in the electrical system used in converting A.C. to D.C. for cathodic protection.

A transformer having variable taps on the secondary winding is used to supply alternating current at the proper voltage for rectification. The a-c voltage applied to the rectifier stack is governed by the resistance to be overcome in the rectifier circuit and, more importantly, between the anodes and the surface being protected; so that an adequate current density is provided to stifle the corrosion currents on the steel in the particular environment being encountered.

Copper oxide rectifier stacks, which have been in continuous operation for more than fifteen years, are reported by the manufacturer to be functioning satisfactorily with very little drop in efficiency from that originally observed. The copper oxide and the selenium type rectifiers are somewhat more efficient in current conversion than are the copper sulphide rectifiers. Their overall efficiency, including the transformer when operating at rated capacity on full wave single-phase rectification, is in the neighborhood of 55 to 65 per cent, depending on the particular characteristics of the rectifying unit assembly.

Over two thousand copper oxide rectifiers, some of which were installed more than six years ago, have been functioning continuously in normal operation with no breakdown of the rectifier stacks. Fig. 7 illustrates a typical rectifier assembly in a weatherproof housing, used in the application of cathodic protection to steel water tanks. Usually these rectifiers are cooled by natural convection of air over their cooling fins, although fans are sometimes used on larger capacity units to reduce the size and cost of the rectifiers.

Motor Generators

Motor generators[18] are frequently used in converting A.C. to D.C. of the proper characteristics for cathodic protection. They are most practical when the output requirements exceed 3,000 watts. Because of moving parts, motor-generator sets require considerable maintenance attention and necessitate proper housing for protection from the elements. Motor generators are usually more efficient than the dry type rectifiers, but their original cost is greater than that of a dry type rectifier of equal capacity.

Windmill and Engine Generators

Windmill generators are sometimes used in pipeline protection where A.C. is not available. They are dependent on continuous wind velocities in excess of 5 mph. An electrical circuit has been designed[19] whereby storage batteries charged by a

windmill generator may be used to provide current during periods of low or no-wind velocity.

Gas engine generators,[15] taking gas as a fuel from the pipeline being protected, are being used on natural gas pipelines in regions where the wind velocity or constancy is not adequate for windmill generators and A.C. is not available for rectifiers. At sewage plants engine generators may well be considered because of the availability of by-product fuel in ample quantity.

Storage batteries, except as a supplement to windmill generators, or the wet type rectifier, are rarely used in cathodic protection practice because of their low efficiencies and high maintenance costs.

Evaluating Protection

Evaluating the degree of protection being derived from any one of the four basic methods of corrosion control, including cathodic protection, is often complex, and such an evaluation should be made by a highly competent engineer experienced in corrosion.

One can be guided by the principles of the electrochemical corrosion mechanism for evaluating the effectiveness of cathodic protection, which was discussed in Part I. There are two phenomena which in practice characterize the corrosion of steel in natural waters; namely, the wastage of metal usually evidenced in the form of pits and the coincidental formation of rust or tubercles.

It was noted that, in the corrosion process, iron atoms leave the crystal lattice of the metal surface and enter the natural water or other corroding media as ferrous ions. This process is physically shown by the development of pits on the localized anodic areas. The pitting type of attack is unfortunate, since many structures can no longer confine liquids when leaks develop even though more than 99 per cent of the original metal in the structure is sound and intact. On this ground, it becomes evident that the rate of attack or pitting of steel in localized areas is often more important than the total amount of metal lost.

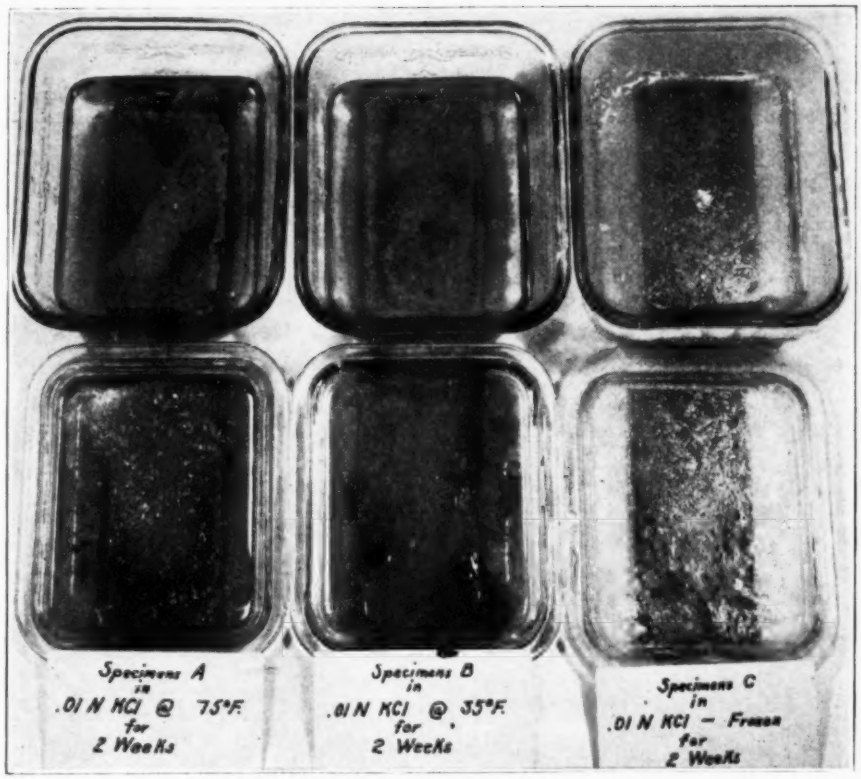

Fig. 8—Test Specimens Used to Study the Rates of Corrosion of Steel at Low Temperatures and the Effect of Ice Coating in Excluding Oxygen and Elimination of Corrosion Behind Ice Formations in Elevated Tanks or Standpipes.

Rust and tubercles, the corrosion products familiar to everyone, are formed by the precipitation and oxidation of the ferrous ion introduced into the corroding media by the natural anodic processes occurring on the steel surface. This rust usually covers the pitted areas and is observed in many forms. In cases where the corrosion process is active, and is under oxygen control, a hydrated black ferri-ferrous oxide (magnetic in nature) is found in the bottom of the pits. As the outer surface of the corrosion nodule is reached, the dissolved oxygen in the water has converted the black magnetic iron oxide to the more highly oxidized form of iron, and is commonly seen as red rust or hydrated ferric oxide.

The volume of rust created by the corrosion process which covers the pitted areas cannot be used as a basis for estimating the depth and amount of pitting, because the volume of the corrosion product (in situ), is usually many times as great as that of the metal from which it was formed. The volume of the corrosion product is governed by the manner and the environment in which it was formed.

In general, the effectiveness of any method of corrosion control can be established by comparing the rate of metal wastage, indicated by metal thickness measurements or by weight loss over a definite period of time, when the corrosion control system is being used against the rate of metal wastage in the absence of the particular corrosion control method.

Determining the Effectiveness of Cathodic Protection

Great care should be taken in the use of corrosion test specimens and interpretation of results therefrom, for evaluating the degree of protection being obtained. Unless the specimen is of the same metal, and has the identical surface condition found on the structure being protected, the same current density will not produce an identical protective effect on both surfaces. Secondary corrosion currents, resulting from the dissimilarity of the test specimen and the surface of the structure being protected, will lead to erroneous results often unfavorable to the method of corrosion control being applied.

Another condition to be met in the proper use of test specimens is that the specimens should receive the same current density as the wall being protected. This may be accomplished by coating one side of the specimen with wax and placing the waxed surface face against the wall. The author, in using this technique for current density distribution studies, found that the test specimen would receive within ten per cent the current density reaching an equal wall area having an approximately identical surface condition providing the potential drop through the current measuring device was compensated for by an opposing potential. Test specimens are usually compared with unprotected specimens in the same environment on the weight loss basis, since the depth of pitting characteristics on test specimens (sometimes called coupons) are rarely comparable to those found on the submerged structure. Test specimens, positioned some distance from the wall of the structure, will usually receive a higher current density than an equal area on the wall. This is true particularly in water tanks. Although test specimens have many limitations, they are helpful in establishing a general picture of a particular corrosion phenomenon.

Measuring the depth of pits at intervals by means of a depth gage resting on corrosion-resistant permanently fixed pins, is one of the most satisfactory methods of establishing the degree of protection obtained, providing the cathodic film around the pit is not disturbed. By disturbing the cathodic layer of calcium carbonate, rust, etc., around the pits, a higher current density will be required to stifle further progress of the corrosion process. Therefore, if a corroded tank is cleaned a larger current input will be required in the early stages to stop the corrosion which would otherwise be accelerated.

Complete protection of a new uncorroded surface is established. if pitting or the accompanying formation of rust is not observed.

The formation of a white deposit consisting mainly of calcium carbonate on the metal surface by the application of cathodic protection is usually clear-cut evidence that the corrosion process has been stifled. The loosening of previously adherent rust and tubercules by the formation of alkali and/or the evolution of hydrogen gas between the rust and metal surface, when a current density greater than that required to reduce the dissolved oxygen

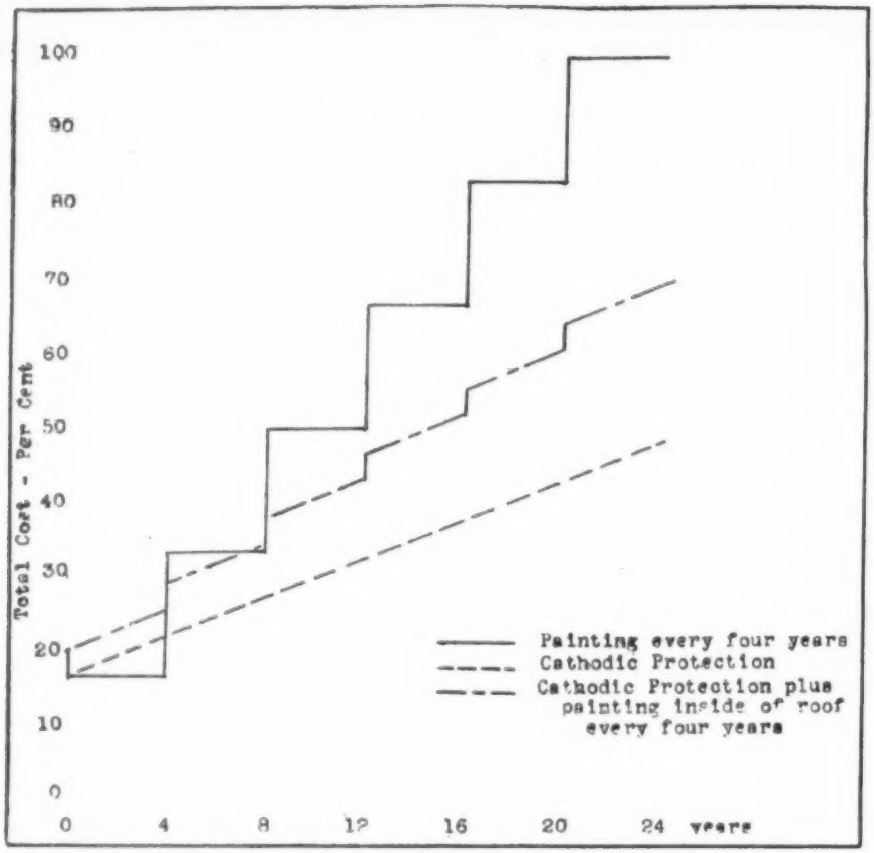

Fig. 9—A Comparison of Overall Costs of Cathodic Protection vs. Painting of Interior Tank Surfaces During a 24-Year Period.

reaching the metal surface is applied, also indicates protection. Often this old corrosion product will be loosened and then slough off the metal surface by its own weight. In the author's opinion, however, this rust layer, if it is continuous over the metal surface, is an effective cathodic layer and will decrease the current density requirements for complete protection. It is therefore a waste of effort and money to clean the walls of corroded tanks, etc., before applying cathodic protection.

"Null"[13] and other electrical methods[14] have been developed whereby changes in metal potential, measured against a standard reference cell with varying projected current densities, may be interpreted to indicate the "minimum current density" requirements for complete protection. Although these techniques are intricate, in recent years they have been widely used in evaluating current requirements for pipe-line protection.

Ewing[14] has found that applying a current density sufficiently great so that an arbitrarily selected potential on a metal surface in respect to a reference electrode is reached, is not always a criterion of complete protection.

Cathodic protection cannot be expected to replace metal already lost by the pitting action or to cause rust formed before the application of cathodic protection to vanish. The main purpose of this method of corrosion control is to retard the rate of attack so that the useful life of the structure being protected will be extended beyond that which can be realized by other means of corrosion control. The reading of an ammeter on a rectifier should not be used as a criterion of protection.

In the protection of petroleum and natural gas pipelines, which have been installed for some time prior to the application of cathodic protection, a comparison of the leaks found each year before treatment against the number occurring after cathodic protection is applied has provided a very satisfactory way in establishing the value of cathodic protection.

Water Tanks

In tentative specifications on the maintenance of elevated steel tanks and other water storage tanks, prepared by a committee of the A.W.W.A. in 1943, it is stated: "Experience has shown that in practically all cases the inside surfaces of a water storage tank are in worse condition than the outside. Almost without exception tank bottoms which have had to be replaced on account of leaks through the steel have corroded through from the inside." From these statements it is apparent that the inner submerged surfaces will demand greater attention insofar as corrosion control is concerned than the outside. It has been the author's experience in examining many tanks that the bowls of elevated tanks and the floors of standpipes are usually attacked much more severely than the vertical walls are. This may be accounted for by the localization of attack on areas under insoluble suspended matter which has settled on the horizontal or inclined surfaces of the tank. For example a leaf covering an area on the floor will cause pitting underneath it, since the leaf masks this area from the dissolved oxygen in the water. The areas around the leaf will be more accessible to the dissolved oxygen in the water and will become the cathode of the oxygen differential concentration corrosion cell, where the depolarizing process controlling the rate of corrosion will take place. Unrelieved stresses resulting from the fabrication of the floor plates may also cause corrosion cells.

For more than eight years cathodic protection has been extensively applied in the mitigation of corrosion of tank surfaces in contact with corrosive waters. Experience and considerable fundamental research conducted during this period has shown that the mere installation of an anode from which current flows according to an arbitrarily chosen fixed current density constant usually will not provide satisfactory protection. In each tank the corrosion problem is unique and the engineering design of a proper cathodic system demands an accurate interpretation of the the corrosion environment. Some of the fundamental factors influencing current density requirements and anode design are discussed in the first article of this series—see Part I.

Some Case Histories

A practical example of the effectiveness of cathodic protection is illustrated by an installation in a 100,000-gallon elevated tank with an ellipsoidal bottom in Dayton, Ohio, which was built in 1931. The tank was initially painted with one shop coat of red lead and then covered with a field coat of red lead after erection. The tank water level was automatically maintained with a fluctuation of about two feet between high and low water. In 1939 several leaks developed in the bowl of the tank resulting in the need for immediate repair. The leaks and most of the pit depressions were repaired by welding. The AWWA specifications[23] previously mentioned state that "such a leak indicates that the entire bottom may be thin and other leaks may soon appear. The use of welding to patch such leaks is of very doubtful economy as the heat of welding very often is the cause of new leaks. Even if a considerable area is welded, it may be but a short time after such repairs are made before new leaks appear."

After repairing the tank bowl in 1939 a cathodic protection system was designed and installed in this tank. An examination of the tank bowl, vertical walls and riser, in the summer of 1944, showed these surfaces to be in very good condition and that the tank should serve without further repair for many years. This particular corrosion problem was extremely interesting since the water confined was from a deep well source containing very little dissolved oxygen and apparently having salts in amounts characteristic of the deep well waters in this mid-western area. However, the dissolved oxygen concentration in the bowl of the tank increased because of contact with the air surface, as will be noted in the analysis below:

Deep Well Water "A"

$Si-O_2$	21	ppm.
Ca (Calcium)	93	ppm.
Mg (Magnesium)	31	ppm.
Na (Sodium)	9	ppm.
Bicarbonate alk.	337	ppm.
SO_4 (Sulphate)	72	ppm.
Cl (Chloride)	20	ppm.
Total Hardness as $CaCO_3$	360	ppm.
pH	7.2	ppm.
Dissolved O_2 in well water	.28	cc./liter
Dissolved O_2 in water in bowl of tank	3.6	cc./liter

A relatively low current density in this case proved to be adequate in retarding the progress of further corrosion and to form a white deposit on the submerged metal surface through the interaction of the $Ca(HCO_3)_2$ in the bulk of the water with the highly alkaline layer on the metal surface produced by the ordered reduction of the hydrogen ions and the dissolved oxygen reaching the metal surface by the protective current flow. Because of the absence of appreciable amounts of incrusting salts in some waters, a higher current density would be required than was applied in the previously described case.

The dissolved oxygen concentration usually is not uniform throughout a tank. Studies made by the author on many tanks have generally indicated that with water supplied containing a low dissolved oxygen concentration, the concentration of dissolved oxygen in the tank will decrease with the depth from the surface, whereas in tanks supplied with water having a high dissolved oxygen concentration, the oxygen concentration will increase with the depth from the surface. This variation in oxygen concentration may affect the intensity of corrosion in the tank at different depths and will affect the current density requirements for complete corrosion control on these areas.

There are three factors which govern the dissolved oxygen content in the water throughout the tank and probably account for the above observations.

1. Oxygen from the air will tend to enter the water through the water surface-air interface.[23] The amount of oxygen dissolved will be governed by surface area, temperature, movement of the bulk of the water, the oxygen already in the water, etc.

2. Oxygen already in the water will be consumed by the corrosion current or if cathodic protection is applied by the projected current, i.e., reduction of the O_2 reaching the metal surface to hydroxyl ions according to the reaction shown early in this article. Aerobic micro-organisms in the water will tend to consume dissolved oxygen. This behavior of the micro-organisms has been demonstrated by the absence of dissolved oxygen in sea water[24] at great depths and in the far ends of water distribution mains.

3. The other factor governing the variation of the dissolved oxygen concentration throughout the tank is the displacement phenomenon which occurs when water enters or leaves a tank. On a humid summer morning one will note, as water is being pumped into the tank, the formation of a distinct boundary ring below which condensation occurs. This ring creeps up the tank wall as the tank is being filled, indicating a distinct line of demarcation between the bulk of the water already in the tank and the cooler incoming water. The author has observed this phenomenon on the risers of elevated tanks and on the walls of standpipes whose diameters have often exceeded 100 feet.

In some cases where the incoming water is very corrosive because of a high oxygen concentration, anodes should be provided in elevated tanks to protect the riser. The author in Part I has shown data indicating that only a very small current density would reach the riser walls from anodes suspended in the bowl of the tank. An anode extending the full length of the riser is often required to satisfactorily protect the riser.

A great deal of emphasis is usually placed upon the corrosion occurring on the intermittently submerged surfaces of a tank. Studies made by the author on a midwestern elevated tank indicate that cathodic protection will cause deposition of $CaCO_3$ and/or $Mg(OH)_2$ on these surfaces during submergence and that this deposit will effectively protect the surface during the period of atmospheric exposure.[20] In waters from which it is difficult to deposit protective scale-forming salts, protection will be obtained only when the surface is submerged and an adequate current density reaches the area. On re-exposure to the air in the absence of an insoluble salt coating corrosion can proceed. However, the corrosion rate during the period of exposure to air is usually negligible compared to that which would occur if the area is submerged in the absence of cathodic protection. In practice it is recommended that the inside of the roof and the wall of the tank be painted down to the mean water level when cathodic protection is applied. The author has already stated that attack on the floor of a tank is usually more severe than that found on the intermittently exposed surfaces of the tank. Therefore, any effective cathodic system must be designed and operated so as to insure maximum protection for the tank bottom and riser pipe.

Ice Interference

In the engineering design of a cathodic protection system some consideration must be given to the formation of ice within water storage tanks and other submerged structures. Ice formation in water tanks is commonly found along the northern fringe of the United States and in Canada where the mean temperature is far below freezing for many months. This ice generally starts to form on the submerged walls of the tank and finally bridges across the surface of the water. Peak thicknesses of 7 to 8 ft. of ice along the tank wall have been reported in Montreal,[25] although thicknesses of 1 to 3 ft. are more common in the United States. The formation of ice in tanks is influenced by the mean temperature outside, frequently of water change in tank, temperature of water in supply system, heat dissipating surface area vs. tank volume relationship, tank covering, location insofar as wind protection, etc. A change in any one or all of the previously mentioned factors may account for formation of ice in one tank, whereas a short distance away in another tank little or no ice formation is encountered.

When the ice formation begins to melt, its behavior is not predictable. Often in breaking up, cakes of ice drop from the walls into the water, sink deeply and then float to the surface. In some cases the intact ice wall may ride up and down the inner tank wall like a piston with changes in temperature and of the water level. Cases have been reported where the upper edge of the ice cylinder will extend several feet above the rim of an open top tank. The movement of the ice tends to scrape the inner wall of the tank, scoring any protective coating. The scoring of a paint film will tend to hasten corrosion and require yearly patch paint application for satisfactory protection. This painting must be done as soon as the ice has melted since the most rapid corrosion on the exposed areas will usually occur during the summer.

Since it is common practice to suspend the anodes used in the application of cathodic protection to water tanks from supports above the surface of the water, it becomes evident that the shifting of ice in the tank may in some cases break or bend the anode assembly.

Much thought has been given to the most satisfactory procedure in applying cathodic protection where ice is prevalent.

Temperature and Corrosion Rates

A study of the influence of the formation of ice upon the corrosion rate of steel is most helpful in establishing the need for cathodic protection during the freezing period. In order to demonstrate this, the author placed two weighed hot rolled steel specimens 4-in. x 2-in. x 1/16-in. in a .01N KCl solution at 75° F., 35° F., and frozen in the solution respectively for two weeks. Fig. 8 reveals the results.

The .01N KCl (potassium chloride) solution used in this study being in contact with air, was extremely corrosive. The weight losses of the two specimens under each of the three conditions of temperature noted above are as follows:

Specimens	Original Wt., Grams	Final Wt., Grams	Weight Loss Grams	Temperature
A-12	60.1546	59.9557	0.1689	75° F.
A-13	60.1574	60.0009	0.1565	75° F.
B-14	60.6673	60.5576	0.1097	35° F.
B-15	60.5917	60.4836	0.1081	35° F.
C-16	60.3867	60.3865	0.0002	Frozen
C-17	60.1246	60.1240	0.0006	Frozen

The rust formed by the attack on specimens A and B may be noted by the presence of nodules of corrosion product along the edges of the specimens and the dark precipitate lying on the bottom of the glass vessels. The uneven surface of the ice accounts for the shadows showing on specimens C. Specimens C were quick frozen in the .01N KCl solution and no corrosion product was noted at the end of the two-week test period.

Even though the solubility of oxygen from the air was greater in the solution around specimens B at 35° F. than in the solution around specimens A at 75° F., the corrosion rate was greater on the latter specimens. This illustrates the influence of the temperature upon the effective thickness of the diffusion layer and the corrosion rate. Speller[11] has shown that the corrosion rate in an air saturated solution increases with temperatures up to about 80° C. Above this temperature the reduction of oxygen solubility in an open top vessel with increasing temperature becomes more apparent in its effect in the reduction of the corrosion rate, so that at the boiling point the corrosion rate is essentially zero.

The extremely high electrical resistance of ice, and its low fluidity which retards oxygen diffusion, results in a very low rate of corrosion attack of the steel coated with ice. From the demonstration it may be concluded that the progress of corrosion is negligible when the submerged surfaces of a steel structure are covered with ice.

In practice this is helpful since the anodes can be removed just before ice forms and reinstalled as soon as the ice melts to provide protection during the period that the metal is in contact with a corrosive water.

In order to eliminate anode removal and replacement each year, heated anodes are being considered along with anodes supported

from the floor of the tank below the ice. However, the use of these ideas is governed by peculiarities of the ice formation and operation of each tank.

Economic Considerations

The prime purpose of any method of corrosion control is to obtain the maximum useful life and the most satisfactory service from the structure being protected at the lowest overall cost. Failure to properly protect the submerged surfaces of a structure such as a water tank will in many cases result in a much greater depreciation rate of the tank than was anticipated at the time it was purchased. The financing and operating problems incurred by the need for early replacement or repair of the tank structure should be guarded against by a planned tank protection program.

The need for a planned corrosion control program applied to water storage tanks is illustrated by a statement found in an AWWA committee report.[28] "Such a leak may ordinarily exist only in the case of a very old tank or one which has not been cleaned or painted on the inside for about twenty years . . ." The author has previously discussed a tank which had leaks developing within eight years after erection. Other tanks, which are unprotected, depending upon the corrosion environment and operating conditions, may operate for more than twenty years without the development of leaks.

With a soundly planned corrosion control program, the life of a water storage tank should be extended to at least fifty years and even longer. Although there are always exceptions, it may be assumed generally that the depreciation rate of a water storage tank structure will be reduced from around five per cent a year to less than two per cent a year by the use of an adequate corrosion control program.

In comparing the cost of painting the inside of a tank against the cost of cathodic protection, certain basic principles should be borne in mind.

(1) Since a properly designed cathodic system will give complete protection of the submerged surfaces, its cost should be compared only against a painting cost in which the best type and grade of materials, labor and technique are used. This should include the proper cleaning of the surface by wire brushing, sand blasting or flaming before paint application; the best grade of red lead and cover coat paint; adequate drying time between the application of the first and second coats and the filling of the tank with water, and the proper application of the paint by a reliable painter. The protective value derived from a paint coating applied to an improperly cleaned and prepared surface or the use of substandard grade paint and labor will not justify the expenses incurred.

(2) If paint is used it is recommended that the tank interior be examined annually,[28] particularly where ice formation is found, and any areas where failures have occurred in the paint film should be patched. Finally, when the paint film has generally deteriorated the entire surface should be properly conditioned and repainted. Experience of the tank manufacturers[29] has shown that repainting is usually necessary every three to five years.

(3) Since the original cost of a cathodic protection system includes equipment such as a rectifier, wiring and other appurtenances whose estimated life is twenty-five years, any comparison of total cost of cathodic protection against painting should extend over this 25-year period. Experience during the past nine years has shown that the maintenance and operating costs of a cathodic protection system on water tanks, including anode replacements and labor and power, rarely exceeds eight per cent of the original cost annually.

Perryman[27] has prepared valuable data which establishes a basis for estimating the cost of protecting steel tanks by various methods. In estimating painting costs he has developed a chart showing the gallons of paint required to cover the inside and outside surfaces of tanks of various sizes and uses, and employs a factor of four times the cost of the paint to be added to that of the paint in order to cover the cost of labor, a nominal amount of wire bushing, scaffolding, etc. The cost of applying two coats of paint to the inside of the tank is about equal to the cost of a cathodic protection system. However, when cathodic protection is applied, the cost of painting the inside of the roof of the tank and the side wall down to the mean water level should be added to the cost of the cathodic protection system. In many cases the inside surfaces above the mean water level have been painted without emptying the tank.

Since the price structures change from time to time, the author has prepared Fig. 9 to illustrate the basic factors previously discussed which are involved in the comparison of the overall costs of protecting water tanks for a period of twenty-four years. The relative cost of painting every four years against the cost of cathodic protection is shown in terms of per cent, the cost of painting being 100 per cent.

Since cathodic protection can be installed and maintained in service without draining the tank, a condition which is necessary when paint is applied, the risk of being out of water in event of a fire or during peak load demand is eliminated. This advantage has not been evaluated but it can be visualized that under some circumstances the cost of a cathodic protection system can be paid for many times because the tank has remained in service during an emergency.

Another factor to be considered is that a paint film is not completely effective as a corrosion barrier during all the period between paintings. As time goes by, the paint film begins to fail so that at the end of three to five years a measurable amount of attack has occurred at the points where film has failed. On the other hand the effectiveness of cathodic protection is continuous so long as the equipment is properly maintained.

Part III—Applications to Hot-Water Tanks, Clarifier Mechanisms, and Deep Wells

Cathodic protection can be successfully applied to any steel structure submerged in natural water providing (a) the submerged surface contour of the structure lends itself to a practical anode design for proper current distribution and (b) an adequate current density is used to stifle the corrosion currents which flow through the corroding media between local areas of different potential on the surface of the unprotected metal. In addition to water storage tanks which are usually simple geometrical structures, cathodic protection is being used on other more complex structures in contact with natural waters. Amongst these are hot water tanks; clarifiers; deep well column pipe and well screens and casings; open tank condensers[30]; revolving screen frames; sludge blanket filtration and Zeolite type softeners; etc. The protection of pipelines in contact with corrosive soils by means of a current flow projected to the exposed metal surfaces is accepted practice by all the major oil and gas companies and some industrial and municipal water systems.[31, 44] The rate of depreciation of many miles of pipeline, originally well covered with coal tar enamel or asphalt coating, where, after a few years, the number of leaks per mile occurring yearly as a result of soil corrosion, often reached prodigious figures, has been reduced by the use of cathodic protection. A relatively small expenditure for a corrosion survey by an expert corrosion engineer, cathodic protection equipment and electrical energy, on a pipeline rendered nearly unserviceable because of soil corrosion, will reduce to reasonable limits the nuisance and loss from frequent leaks, and the need for ultimate relaying of all or a portion of the pipeline. The application of cathodic protection to sewage equipment is a relatively new field in which much work is under way as recorded by Parkes[44] and Kozma.[45]

The Hot Water Corrosion Problem and Cathodic Protection Experiences

Hot water tanks, such as those used in domestic, institutional and industrial water systems, provide an excellent illustration of some of the corrosion fundamentals that must be considered in cathodic protection design. Frequently the only serious corrosion problem encountered in a municipal water system is that found in domestic hot water tanks. It has been reported that over 500,000 domestic and commercial hot water tanks are replaced yearly in the U. S. because of corrosion. It can be seen that because of the magnitude of the problem those responsible for water systems are actively seeking methods for alleviating this corrosion. Many methods of corrosion control for hot water tanks have been tried and adopted. Amongst them are use of inhibitors; controlled lime saturation; deaeration; linings of cement mortar, glass, enamels or synthetic resin coatings; metallic coatings such as zinc, which often provide protection only for a limited period; and cathodic protection.

A general picture of the cause and characteristics of corrosion in hot water tanks using municipal water supplies may be obtained from a study of up-to-date power plant water conditioning practice. In order to prevent corrosion in a boiler feed water system, the water before passing through the piping and stage-heaters on its way to the boiler is usually deaerated in a mechanical degasifier where the water is exposed to favorable temperature and pressure conditions. Following this treatment sufficient sodium sulphite, or other oxygen scavenging chemical, is added to the water to reduce the traces of oxygen not removed by the deaerator. In spite of feedwater temperatures as high as 400° F., a pH as low as 6.5, the presence of dissimilar metals in heat exchangers, soluble salt concentrations ranging from 0.5-200 ppm. and the absence of scale-forming salts or inhibitors, the corrosion or pitting rate in feedwater piping, heat exchangers and tanks in the feed water system is negligible so long as *the dissolved oxygen is completely removed*. Since microorganisms

can hardly contribute to the corrosion process in hot water tanks, and since other oxidizing agents or depolarizers usually are not present in appreciable amounts in municipal water supplies, it becomes apparent that the factor governing this corrosion process is the presence of dissolved oxygen. At the present time it does not appear generally feasible to deaerate water distributed in municipal water systems because of the cost of deaeration, the possibility of encouraging the growth of anaerobic microorganisms and degradation of organic matter in the mains, and the presence of a flat taste found in the water when the soluble gases other than dissolved oxygen have been removed.

In a sense cathodic protection may be likened to an electrolytic deoxygenation process concentrated immediately on the protected metal surface. This thought will be illustrated later on in this paper by the reduction of the concentration of dissolved oxygen in the water leaving the hot water tank compared to that entering, and without any wastage of the steel tank being noted. Speller[34] has used the "oxygen drop method" as a direct measure of the amount of corrosion which has taken place in a natural water environment. By measuring the rate of flow and the decrease in oxygen content of the water leaving a certain section of the water system against that of the water entering, the amount of corrosion attack occurring in the section can be closely predicted. When properly designed cathodic protection is applied there is no metal wastage of the steel tank corresponding to the oxygen drop. It is evident therefore that the oxygen has been reduced without the flow of local corrosion currents on the metal surface.

Columbus, Ohio, Experiments

During the past few years a number of hot water tank corrosion experiments have been conducted by C. P. Hoover[35] of Columbus, Ohio. Amongst these tests, the objective being an evaluation of the many methods of corrosion control, was the application of cathodic protection to two of the battery of 30-gallon domestic hot water tanks. One of these two tanks was fabricated of black iron, the other of galvanized iron. The cathodic protection equipment for these tests, which began on June 9, 1941, and were completed on June 9, 1943, was provided by the E. R. P. Corp. The tanks were operated during this interval at a temperature between 140°-150° F. with ten gallons of water being withdrawn from each tank three times daily the first year and twice daily during the second year of the test. Treated Columbus city water used in this test is usually close to saturation with oxygen and has a positive lime saturation index. In the cathodic protection 0.6 and 0.35 amperes were applied to the galvanized and black iron tanks respectively. Fig. 10 illustrates the anode position in respect to the tank. Average oxygen concentrations measured in the effluent of the galvanized and black iron tanks were 1.9 ppm. and 2.4 ppm. respectively. At the end of two years the tanks were cut open and inspected. The observations made were very interesting. In both tanks protection was complete to within ½ tank diameter below the anode. The protected surfaces were covered with a hard thin scale having the following analyses:

Tank Deposit Composition

	Black Iron Tank	Galvanized Tank
SiO_2	3.9%	6.0%
Al_2O_3	4.7%	31.1%
Fe_2O_3	43.2%	10.4%
P_2O_5	0.3%	0.1%
CaO	14.6%	3.8%
MgO	8.1%	0.8%
ZnO	2.9%	30.2%
Ignition Loss—CO_2	11.0%	17.0%

Below this line of demarcation a few pits on the lower 12 inches and bottom of the tank were noted. On re-examining Fig. 10 the reason for this phenomenon can be discovered. The probable current density distribution along the wall of the tank is indicated by distance of the c.d. (current density) line from the wall of the tank. It will be noted that the current density on the tank wall below the electrode decreases rapidly until it is no longer adequate for complete protection. Even so, partial protection is received below the region where the minimum current density requirements are being provided. The walls opposite the electrode received a much higher current density than was actually required for complete protection.

Many objective studies of cathodic protection on practical applications such as this coupled with a basic background on the fundamentals of current density requirements and anode design now provide a more satisfactory treatment of this corrosion problem. For instance, the need for a longer electrode, to provide a more nearly uniform distance of current travel from anode to the surfaces, requiring protection, was illustrated in this test. This case further illustrates ...e importance of providing the proper current distribution which factor is influenced only by the potential relationships existing between the electrode configuration in the corroding media and the submerged surfaces of the structure being protected. It will be noted that neither galvanizing nor more treatment productive of a scale forming waters can be depended on to provide adequate protection where dissolved oxygen is present. Particularly is this fact evident where hot water corrosion is involved.

Public Service Electric & Gas Company

A cathodically protected 300-gal. hot water generating tank of ¼-in. steel plate at the Kearney, N. J., generating station of the Public Service Electric & Gas Co. has been previously reported on by E. H. Thwaits[36, 37] of the Am. Iron and Steel Inst. The present tank was installed in April, 1942, to replace a tank which had been patched repeatedly because of leaks developing after but fourteen months of operation. In the heater tank 13.5 square feet of copper steam heating coils which are insulated from the main body of the hot water tank are located near the bottom of the horizontal tank. The area of the coils is adequate to heat the volume of water contained within the tank from 60° F. to 180° F. in one hour. In the unprotected tank the pitting was found to be nodular at the top of the tank and, as the wall was examined from the top downwards, it was noted that the corrosion product oozing out of the lower nodular anodic areas streamed down the tank wall like a veil. Under this veil of corrosion product, which masks the metal underneath from the dissolved oxygen in the water, further pitting action was initiated. When the veil-like corrosion product was removed from the wall, long finger-like pits were revealed. The greater intensity

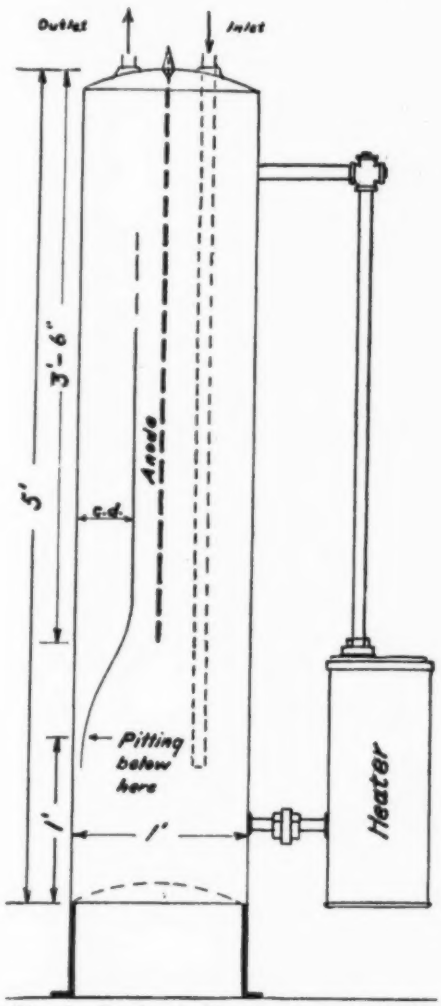

Fig. 1—Sectional Sketch of Cathodically Protected Domestic Hot-Water Tanks, Columbus, Ohio, Studies.

(Adequate protection extended over the wall area below the terminal of the anode only for a distance of one-half the tank diameter—adequate proof of the need for effecting approximately equi-distant current travel from anode to the protected surfaces, if economically effective protection is to result.)

of corrosion noted near the top of the tank probably can be accounted for by the turbulent effect created by the convection current rising from the heating coil striking the top and sweeping down each side of the tank.

The water heated in this tank is Wanaque Reservoir tap water having the following analysis:

Content	ppm.
Temporary hardness	16.3
Permanent hardness	11.1
Total dissolved solids	51.5
Suspended matter	.86
Silica	6.5
NaCl	6.0
M. O. alkalinity	18.0
pH—adjusted by treatment	8.8

Starting in April, 1942, a c.d. of about 1 amp. per 2,000 sq. ft. was applied to this tank and no leak had developed after about 22 months of operation. Although the progress of the corrosion process had been reduced below that which had been previously encountered since, it was clear that corrosion had not been completely stopped as evidenced by the continued formation of rust and the presence of pits. In January, 1944, the current density was increased to 1 amp. per 750 sq. ft., but it was simultaneously revealed through research then being conducted at the Westport Mill of the Dorr Co. that even a higher c.d. requirement was indicated for this tank. In the spring of 1944 a larger rectifier was supplied in consequence and a current density of 1 amp. per 35 sq. ft. (2.86 amps./100 sq. ft.) was applied.

Three months later, upon re-examining the tank, the corrosion product was entirely removed and the surface of the tank was found to be coated with a light powdery-like deposit. The absence of active attack was striking. In December, 1944, the current density was reduced to 1 amp. per 70 sq. ft. (1.43 amps./100 sq. ft.), and in view of the particular corrosion environment it is the author's opinion that this c.d. will provide satisfactory protection. Tests made of the oxygen content of the water entering and leaving the tank were as follows:

Water entering the tank, 13.95 ppm. O_2
Water leaving the tank, 5.85 ppm. O_2

Therefore, the reduction of dissolved oxygen content of the water occurring in tank was 8.1 ppm.

Discussion

In the absence of cathodic protection the actual metal wastage from the tank walls could be predicted from a consideration of the amount of water withdrawn and the reduction of the dissolved oxygen content occurring within the tank. For example the reduction of 8.1 ppm. of dissolved oxygen in 300 gallons of water passing through the tank would represent the degradation of about 21.4 grams of metallic iron from the tank wall, this iron being converted to ferric hydroxide.

The dissolved oxygen in the raw water entering a pressure type of heater will remain in solution except for that which is reduced by the natural corrosion currents (or by the projected current where cathodic protection is applied) since the pressure within the heater will not permit the release and removal of the dissolved gases from the water to the atmosphere.

The current density required to protect an open type hot water heater is usually much less than that required to protect a pressure type of heater. The reason for this will be clearer when it is remembered that the solubility of oxygen and other gases in water at atmospheric pressure decreases with increasing temperature. Therefore, in an open type heater, which is usually vented to the atmosphere, the gases are gradually evolved until they are almost entirely removed at the boiling point. Under conditions where the incoming water is saturated with oxygen it can be seen that the oxygen will remain in the water in a pressure heater whereas in an open type heater a large portion of these gases would be vented to the atmosphere. The heated water thus becomes progressively less corrosive.

Since it has been previously shown that with all other conditions equal, the current density requirements vary with the dissolved oxygen concentration of the water, it becomes apparent that an open type heater will require a lower c.d. than a pressure type heater employed to heat the same water.

New York Central Railroad

A cathodic protection installation in an open type heater at the Harmon, N. Y., yards of the New York Central Railroad has been recently described by E. H. Thwaits.[24] A live steam jet was used to heat the water. It is apparent the live steam used for heating would strip most of the dissolved gases, including oxygen, from the water and, therefore, require a low c.d. for protection of the steel tank. Actually a c.d. of 1 amp./1280 sq. ft. proved adequate for protecting this tank.

Well Pump Corrosion and Cathodic Protection Experiences

The corrosion occurring on the casings, screens and pumping equipment used in deep wells offers a tantalizing problem to those interested in cathodic protection. The failure of the well casing and screen by corrosion usually necessitates the drilling of a new well. Corrosion occurring in the pump bowl, impeller, column pipe and shaft tubing often causes the pump to be pulled prematurely for replacement or repair of the affected parts. There are two types of corrosion currents which influence the attack on deep well equipment. The first is the local corrosion cell type of attack discussed in the author's preceding articles. The second is the man or mechanically created corrosion current which is covered by the broad term—electrolysis. The latter is occasionally found where nearby DC power systems are grounded and current is forced to flow through the water or soil around poorly connected metal joints within the well or to flow from the well through the soil to some adjacent structure possessing a lower potential. Some thought has also been given to the possibility of a rotating pump shaft cutting magnetic lines of force generating direct current which would flow through the shaft and return through the column pipe, etc.

Electrolysis can usually be overcome by a complete survey of the electrical environment surrounding a well and then providing measures indicated by the survey to prevent the forced drainage of current from any metal surface in the well. The local corrosion current cells on the metal surface can be stifled by a projected current from properly positioned anodes within the annular space between the shaft and the column pipe and between the column pipe and the well casing. Other electrodes can be positioned to provide a protective current flow to the inner surfaces of the casing and screen below the pump suction. The corrosion cells caused by dissimilar metals are stifled by cathodic protection. The outside of the casing may be protected against soil corrosion by projecting a current flow through the soil to these surfaces. In view of the complex nature of the interior of the pump bowl and impeller the author does at this juncture not feel that these intricate surfaces can be reached by projected current flow from any practical anode design. This part of the well can best be fabricated of corrosion resistant metals.

Application Experiences

F. P. Macdonald[25] has discussed an oil lubricated deep well turbine pump at La Peer, Mich., whose column pipe was severely pitted when pulled and examined in 1941 after eighteen months of operation. At that time the column pipe was renewed and cathodic protection was applied. Upon re-examining the column pipe after 31 months of operation there was no evidence of attack occurring during that interval. Four other cathodic protection installations in deep wells have been made in Indiana, Louisiana and British Guiana. While these installations are being carefully followed it is yet too early to justify any statement.

Clarifiers and Sludge Blanket Filtration Devices

An unexpected corrosion problem being discovered in most water plants treating raw surface water is the attack of clarifier mechanisms and of the sludge blanket filtration type softeners. This attack is usually not anticipated since in the softening process the water is treated with lime and generally has $CaCO_3$ depositing properties which have been assumed to provide protective properties to submerged steel surfaces. In spite of the positive $CaCO_3$ Saturation Index the corrosion process is active in the presence of such aerated treated water. The freely corroding test specimens shown in Fig. 2 illustrates the type of attack occurring on a water clarifier mechanism in Ohio. The cathodically protected specimens supported against the mechanism structure, shown in the same photograph were completely protected without any corrosion or pits developing during the interval of the test.

An illustration of the type of attack occurring on the submerged surfaces of the Sludge Blanket Filtration type of softener is shown in Fig. 11. The nodular formation of corrosion product (tuberculation) lies over the anodic areas where pitting has progressed beneath these nodules—in a few places even completely through the steel sheet baffles. Cathodic protection is now being applied to many clarifier mechanisms and Sludge Blanket Filtration Type Softeners with very encouraging results.

Dangers in Cathodic Protection Are Few But Require Consideration

The possibility of deleterious effects resulting from properly designed cathodic protection is negligible. It has already been stated in Part I that an excesssive current density will not prove to be harmful to iron. A design consideration that must be made in any cathodic protection installation is the provision for a con-

tinuous low resistance path back to the rectifier from all submerged surfaces receiving a protective current flow. The importance of this is illustrated by the corrosion induced on a poorly bonded pipe joint when current is flowing through the pipe. A portion of the current will leave the pipe and flow through the electrolyte to the adjoining pipe section around the resistance at the joint. Corrosion attack is induced at the pipe joint where the current enters the electrolyte, whereas the adjoining pipe section receiving the same current flow from the electrolyte is being protected. Eliassen and Goldsmith[39] have shown under controlled conditions that grounding of 1.25 amperes of either alternating or direct current at one end of a length of ¾-in pipe and the drainage of this current from the other end of the pipe had no appreciable effect on the quality of the water passing through the pipe. This would indicate that a good current drainage path back to the rectifier from the structure being protected will not influence the normal corrosion processes occurring on surfaces which do not receive a projected current through the corroding media to the metal surface.

Although the fundamental principles of cathodic protection have been well understood for many years and are available in the corrosion literature, the water works field has been fortunate in having had the application of this method of corrosion control to water plant equipment studied and discussed by O'Brien[40], Mabee[41], Norman[42], Knudsen[31], Keith[43], Schneider[44] and by numerous others who have not published their findings. Their observations have contributed much to the better understanding of cathodic protection amongst the men who must battle corrosion in the water and sewage works field.

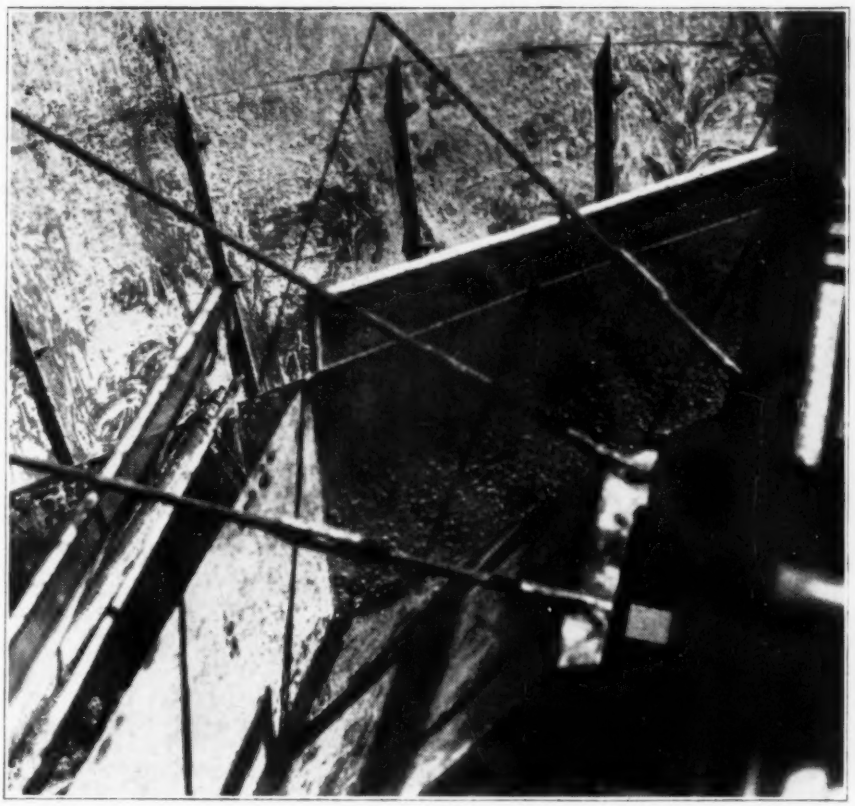

Fig. 3—Metal Work of Sludge Blanket Up-Flow Type of Water Softening Tank (Tuberculation, and pitting beneath these tubercules proceed despite a lime treated depositing water which has not prevented dissolved oxygen attack.)

Conclusions

The application of cathodic protection frequently presents a complex engineering design problem not fully appreciated by many. And, for truly successful results and economy a comprehensive survey of the corrosion environment is the only recommended procedure, except in the simplest cases wherein success seems almost automatic.

Any steel surface submerged in natural waters may be protected when an adequate current density, projected from properly designed anode configurations within the corroding media (and this is of highest importance) is properly distributed so as to stifle the corrosion currents flowing between local cells on the metal surface requiring protection.

All other conditions being equal, the current density requirements for complete protection are a function of the dissolved oxygen content of the water, temperature and velocity.

The deposition of $CaCO_3$, or the existence of other relatively continuous protective coatings on the metal surface, will reduce current density requirements for complete protection.

In regions where ice formation occurs in tanks there is a "corroding season" and an "inactive season." The "corroding season" occurs during the months when the metal is in contact with water, whereas the "inactive season" is the period where the metal surface is protected by an ice film barrier. Protection must be provided during the "corroding season."

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OPERATING LOGS FOR SMALL WATER PLANTS

IN MOST states the department of health requires that monthly records of operating and analytical data be forwarded to the state office and forms are provided for that purpose. In cases where such forms are not provided or where the individual operator desires to keep his own record it may be desirable for the operator to develop a form. Herewith as a guide are three examples of forms used by three different state departments of health.

Concerning the need for and use of operating logs of this type, Charles R. Cox, Chief, Bureau of Water Supply, Division of Sanitation, N. Y. State Dept. of Health, says, "One is likely to neglect the maintenance of records unless their true significance is realized. Actually they represent a summary of work accomplished and of results secured, and, therefore, serve as a guide to future action as well as a proof as to the quality of water delivered in the past. Such records also furnish local and state departments of health with a summary of prevailing conditions for their guidance in the general supervision of public water supplies insofar as the health of the consumers is concerned.

"Actual experience has indicated that well maintained and complete records have enabled water supply officials to demonstrate to epidemiologists studying an outbreak of disease, which is suspected to be water-borne, that the operation of their plant was satisfactory and that water of safe sanitary quality was delivered during the period of onset of the disease. In the absence of such complete records, the scattered distribution of cases might implicate the water supply as being the obvious factor in that it is usually common to all of the cases. The importance of the situation is that evidence could not be confuted in the absence of records as proof of the satisfactory quality of the water delivered during the critical period in the past. Note the significance of the phrase 'in the past,' because no amount of analytical work during an epidemiological investigation can disclose the quality of water long since consumed in the municipality before the onset of the epidemic.

The Chief Value of Log Keeping and Records

"This more obvious value of records of past accomplishment, however, should not submerge the chief value of appropriate records for the guidance of the operators themselves. The mere recording of summarized information by an operator results in his own appraisal of his work, which otherwise might not be considered in such an abstract manner, and the maintenance of records encourages keener observation of operating conditions, frequently resulting in economies along with improved results. Records would be of limited value unless they are used as a basis of judgment and to stimulate keener observation. For example, the mere recording of water temperature in and of itself is of little value, but if the operator knows from past records that when water temperatures are below, say, 39° F., the entering water tends to stratify and flow along the bottom of coagulation basins, thus leading to short circuiting and agitation of previously deposited sludge, the temperature observation becomes of paramount importance. In the same way previous temperature observations may indicate proper coagulation operations technique, or cold weather modifications in filter washing technique. These record blanks are not merely a collection of figures but are summaries of observations which are definitely related to experience and thus serve as guides to future action.

"The basic characteristic of the typical record blank should be the provision for recording three groups of observations: first, plant operating details; secondly, doses of various chemicals used, and thirdly, results of tests as to the character of the water being treated and of the final effluent produced.

Value of Operator's Diary

"The monthly record blank should be supplemented by a diary and the conventional bookkeeping records as to expenses incurred in the operation of a treatment plant. A simple diary is of fundamental value for recording intangible and frequently highly personal items which may be so obvious at the time that their future value may be overlooked. Such information, however, is soon forgotten or becomes sufficiently vague to be of little value unless brief informal notes can be consulted. Diary records are of special value when they indicate procedures followed in solving unusual problems.

"In summary, a habit of mind should be cultivated of viewing the readings of meters, gauges and other instruments and also the results of laboratory tests as visible evidence of what otherwise is not apparent to the operator and for such observations to be recorded for convenience in appraising their joint significance and to serve as a guide for the future."

Patterns of Record and Log Forms

The New York State form is made to fold in the middle to produce a file size of 8½ by 11 inches. In Fig. 1 is shown half of this form covering mainly plant operating data, chemicals used, and some analytical results. The other half of the form (which we regret could not be reproduced because of space limits) contains columns for recording pH, total bacteria and coliform bacteria in the raw, filtered and chlorinated water. On the back of the form are spaces for additional analytical data collected during the month to show the character of water delivered to the consumer or to give more information concerning unusual conditions at the plant. Items included are turbidity, color, odor, hardness, alkalinity, iron, manganese, chlorides, residual chlorine, dissolved oxygen, and oxygen consumed.

Fig. 2 shows a similar form used in the state of Ohio. This form contains more analytical data and fewer entries of plant

To New York State Department of Health:

Report on Operation of WATER PURIFICATION PLANT at ....for Month of ..... 193 ..

Date	Water Treated 1000 gallons	Filters		Wash Water		Alum	Lime	Soda Ash	Carbon	Ammonia	Chlorine				Temperature	Turbidity p. p. m.		Color p. p. m.		Odor	
											Pre		Post								
		Units in use	Units washed or scraped	Total 1000 gals	Length of wash min.	Lbs per day	Lbs per day	Lbs per day	Lbs per day	Lbs per day	Lbs per day	Resid Chlor	Lbs. per day	Resid Chlor	Raw Water	Raw	Filt.	Raw	Filt	Raw	Filt.
	1	2	3	4	5	6	7	8	9	12	13	14	15	16	17	18	19	20	21	22	23
1																					
2																					
3																					
30																					
31																					
Total																					
Aver.																					

Fold Here

48 REMARKS: Filters out of service; complaints (Nature, cause, remedy); Results of microscopic examinations, Etc.

	Date	
Sedimentation Basin last cleaned		
Clear water Basin last cleaned		
Storage Reservoir last cleaned		

Fig. 1. First Half of Monthly Operating Log and Report Sheet of the New York State Health Dept. (This half contains all data on plant operation and chemicals used, etc. Elsewhere in the form are columns for pH and total and coliform bacteria in raw, filtered and chlorinated water.)

Report of Operation to the Department of Health, State of Ohio, for Water Purification Plant at _____

DATE	WATER TREATED Thousands Gallons	FILTER OPERATION: Number Used	Filter Hours	Length of Run	Number Washed	CHEMICALS USED: Gr. per Gallon: Alum	Lime	Soda	Parts per Million: Carbon	Ammonia	Chlorine	CHEMICAL RESULTS IN PARTS PER MILLION: Turbidity: Raw	Applied	Alkalinities: Raw: M.O	Settled: M.O	Phenol	Carbonated: M.O	Phenol	Filtered: M.O	Phenol	Hardness: Non-Carbonate: Raw	Filt	Total: Raw	Filt.	pH: Raw	Filt	Color: Raw	Filt	Magnesium: Raw	Filt	Residual Chlorine: Plant Tap	Distribution
1																																
2																																
3																																
30																																
31																																
Totals																																
Avg																																

Percent Wash Water Used During Month _____

Cost of Chemicals per Million Gallons _____

Month _____ Year _____

_____ Supt.

_____ Operator

Fig. 2. Form of the Ohio Dept. of Health for Water Plants. (Showing operating data, chemicals used, and analytical results.)

operating data and chemicals used. This form is augmented by a second, on which is entered all of the bacteriological test data. The Ohio Department of Health also has a third form used only for zeolite water softening plants. The general idea of this latter form is a combination of the other two with only columns pertinent to water softening included.

Concerning these forms, Thomas R. Lathrop, Assistant Engineer of the Ohio State Department of Health, writes, "Some of the larger plants keep more complete data on operation in their own files. For smaller plants, however, the data covered by these forms are sufficiently comprehensive. Sometimes it is necessary to change column headings to fit special conditions. All forms are 8½ by 11 inches, which is good filing size. Operating log forms are not a fixed standard and often find need of revision at intervals, perhaps to add a new test or delete an old one. Climatological and water temperature data should be included at every plant. Not many plants use all of the columns provided on the Ohio forms, and hence by change of heading may add some test or observation not included."

Fig. 3 shows a typical bacteriological data log and report sheet as used by the Illinois Department of Health. In addition to this form there are three other forms of the same size (9¼ by 14½ inches), and these forms record data similar to that given in Figs. 1 and 2 (the New York and Ohio forms).

About these Illinois forms, William J. Downer, Acting Chief Sanitary Engineer of the Illinois State Department of Health, writes, "The form containing the plant operating, chemical operating and analytical data is used for surface water supplies having conventional treatment and for lime-soda water softening regardless of whether a ground or a surface supply. Another similar form, with slightly different entries, is used for ground water supplies when iron removal, zeolite softening or a combination of the two is involved. Another form records information on well supply plants that have no treatment but chlorination. It was designed chiefly for use in state institutions but has found favor in several public water supply plants. On it are recorded hourly residual chlorine determinations daily, weight of chlorine cylinder, and amount of chlorine used each day as well as volume of water treated each day."

In General

In the preparation of any form, certain ideas and factors should be borne in mind. Size of sheet and ease of filing for ready reference, columnar headings only for items requiring regular entry, provision of an extra blank column or two for miscellaneous data or results, and a column for remarks. (The latter were trimmed out in the accompanying cuts in order to preclude too great a reduction and to conserve space.) Provisions for all data and entries which may properly be included under the three categories of data, set up above, should be made. The columnar headings should be self-explanatory and separated by vertical lines of varying weight to segregate related data into sections. Other general ideas expressed by A. A. Hirsch on a log for a sewage plant, which appeared in the January, 1943, issue of WATER WORKS AND SEWERAGE, have been adapted to forms for a small water plant, and appeared in the November, 1943, issue of WATER WORKS AND SEWERAGE.

TO: DIVISION OF SANITARY ENGINEERING, STATE DEPARTMENT OF PUBLIC HEALTH, SPRINGFIELD, ILLINOIS.

BACTERIOLOGICAL REPORT ON WATER SUPPLY OF _____ FOR MONTH OF _____ 194____

DAY	BACTERIA per ml. (Ager at 37° C.): Raw 50	Appl. 51	Filt. 53	Finished 53	Tap 54	COLI-AEROGENES: RAW (24 hr. Presump. or conf.): 10 ml. 55	1 ml. 56	0.1 57	0.01 58	0.001 59	FILTERED 10 ml.: Presumptive 24 hrs. 61	Presumptive 48 hrs. 62	Part Conf. 63	PLANT—FINISHED: 50 ml.: Presumptive 24 hrs. 64	Presumptive 48 hrs. 65	Conf. 66	10 ml.: Presumptive 24 hrs. 67	Presumptive 48 hrs. 68	Conf. 69	DISTRIBUTION—SYSTEM: 50 ml.: Presumptive 24 hrs. 70	Presumptive 48 hrs. 71	Conf. 72	10 ml.: Presumptive 24 hrs. 73	Presumptive 48 hrs. 74	Conf. 75
1																										
2																										
3																										
30																										
31																										
BACTERIA AVE.																										
MAX.																										
MIN.																										

FOR ADDITIONAL REMARKS USE REVERSE SIDE _____

REPORT BY _____ (TITLE) _____

Fig. 3. Form of the Illinois Dept. of Health for Recording and Reporting Bacteriological Results. (Other Illinois forms are similar to those of New York and Ohio.)

QUICK FREEZING OF SMALL PIPES FOR REPAIRS

By JOHN L. FORD*

WHEN curb valves on water services have to be replaced or when street widening makes it necessary to lengthen the service to the curb valve, it is a big advantage to be able to shut off the pressure. Shutting off the corporation cock at the main generally involves too much digging and expense to be practical unless it is the only way.

In some water works it has been common practice for many years to temporarily freeze water in pipes to serve as a plug in the line and prevent flow during valve changing or other repair and extension work. I recall my father telling how he used a box about 4 in. square and a foot long with notches in the end to slip under service pipes and then fill with ice and salt. The principal requirement for service pipe freezing is stopping the flow during the process, or at least cutting it down to a rate which will permit freezing. In replacing defective curb valves it is generally possible to cut the flow down far enough.

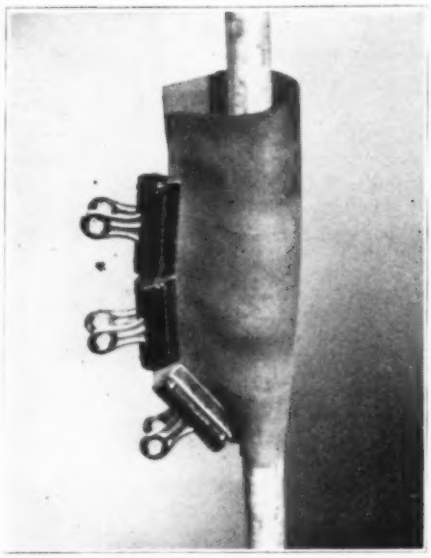
Considering the simplicity and ease of service pipe freezing it has always been surprising that it is not used oftener.

The use of dry ice (solid carbon dioxide) for pipe freezing probably dates from the time when this material first became available. Its principal advantage is that it is so cold that it works amazingly fast. Also it is clean to handle and is now easily obtainable from almost any frozen vegetable distributor or ice cream shipper.

Simple Method of Freezing Service Lines

Here is a quick way to freeze water in pipes with dry ice. The procedure is certainly simple and fast. A piece of old inner tube about 8 in. by 10 in. is centered under the pipe and then brought up and fastened together with wide and strong paper clamps. The rubber should not be stretched tight from end to end but should be a little loose so as to form a pocket around the pipe—see cut.

For freezing a ¾ in. pipe a piece of dry ice about as big as a tennis ball and weighing about one pound is ample. This is placed in a cloth and then pounded into "snow" with a hammer, board or stone. The "snow" is then poured into the rubber pocket and about one-forth pint of alcohol is poured in on top, the two ingredients making a wet mush which is so cold and makes such good contact with the pipe that the water inside is quickly frozen. Any alcohol, anti-freeze or even rubbing alcohol can be used. Gasoline or naphtha are also satisfactory though these may be hard on the rubber. However, old tire tubes can be had for the asking at most service stations so the cost of the rubber makes no difference and gasoline may be more readily available than alcohol.



Suggested Arrangement for Freezing Vertical Pipe

(To make liquid tight joint around pipe press clay, chewing gum or wax into corner between pipe and bottom clamp.)

When the flow in the pipe can be stopped entirely this procedure will freeze the water in a ¾ in. steel pipe solid in from two to five minutes. With a flow of 1/30 gpm. (about one-fourth pint per minute) the flow will stop in about the same time. This flow is more than a drip, coming out of a smooth ended small faucet or small pipe as a steady stream about 1/16 in. in diameter and breaking into drops some three inches below. The experiments on which these results were observed were made with water of about 65° F.; colder water would freeze a little faster. It takes several times as long to freeze a service with dry ice when no liquid is added. Apparently this is because the liquid makes better contact with the pipe and conducts the heat away faster.

In actual work in the trench there may be a small drip from the valve which can be watched to see when the flow stops to indicate that the pipe is frozen.

When there is no leak or drip to watch, the best procedure is to leave the valve closed for five minutes after the dry ice and liquid are poured around the pipe and then open the valve cautiously, keeping in mind that there will be a small amount of water under pressure between the ice plug and the valve which may deceive the operator into thinking the ice plug is not yet solid. If the flow continues, close the valve again before the water thaws much of the ice that has formed, and let the freezing action continue a few minutes longer and again try for flow.

After the pipe is frozen solid it will remain so as long as there is any dry ice left in the envelope and more can be added if desired. After the necessary piping changes have been made the rubber is removed and warm water can be poured on the pipe to thaw it, or a torch can be used. The rubber can be re-used many times and the expense of this method is only 15 cents for material. The time required is only a few minutes in most cases. There is no damage to any kind of pipe by this process, but of course dry ice should never be handled with the naked hand.

If the flow cannot be reduced to about half a pint per minute the pipe cannot be frozen as mentioned above because the warm water passing through keeps a small opening melted through the ice. A longer pocket providing more contact would doubtless freeze a greater flow but it is obviously impossible to stop large flows unless means are available to reduce the leak to a rate that will freeze within a reasonable time.

A Warning

A word of warning should be added about the use of dry ice. The carbon dioxide given off is not poisonous but it can cause suffocation and even death. The small amount used in pipe freezing and the natural ventilation available makes the danger in this work almost negligible. There is little fire hazard with alcohol or gasoline during the freezing because the concentration of carbon dioxide and the low temperature of the liquid prevent ignition.

*Secy. & Treas., The Ford Meter Box Co., Wabash, Ind.

(1) Pound the dry ice into "snow" in a cloth; pour it into rubber pocket around pipe.

(2) Pour about one-fourth pint of alcohol or gasoline on the "snow" and stir to mix.

(3) After 3 to 5 minutes the water in pipe is frozen solid; the repair can be made.

SELECTION OF WATER METERS FOR CUSTOMER SERVICE

By S. F. NEWKIRK, JR.
Engr. and Supt., Water Board, Elizabeth, N. J.

THERE are 28 different types of fluid meters, according to the A.S.M.E. classification, distributed under two divisions and eight classes, as shown in Table I. In this discussion, however, only those types which are almost universally used in the water works profession for the measurement of supply to customers will be considered.

Basically, there are only two classes of meters used for general water customer service—the positive, volumetric type and the inferential, current (kinematic) type.

TABLE I

A.S.M.E. Classification of Fluid Meters

Division	Class	Type
QUANTITY METERS		
1. Positive	1.1 Weighing	1.11 Weighers
		1.12 Tilting traps
	1.2 Volumetric	1.21 Tank
		1.22 Piston
		1.23 Disc
		1.24 Rotary
		1.25 Bellows
		1.26 Wet drum
2. Inferential	2.1 Current (kinematic)	2.11 Propeller
		2.12 Anemometer
		2.13 Turbine
		2.14 Helical
RATE OF FLOW METERS		
	2.2 Head (kinetic)	2.21 Venturi
		2.22 Flow nozzle
		2.23 Thin plate orifice
		2.24 Pitot
		2.25 Centrifugal
		2.26 Friction
	2.3 Area (geometric)	2.31 Gate
		2.32 Orifice & plug
		2.33 Cone & disc
		2.34 Cylinder & piston
	2.4 Head-area (weir)	2.41 Rectangular notch
		2.42 Triangular notch
		2.43 Special notch
	2.5 Force	2.51 Hydrometric pendulum
		2.52 Vane
	2.6 Thermal	2.61 Electric

Compound meters, both for general service and for fire service, are really combinations of these basic types.

Selecting Meter Types

Displacement meters, which are practically positive in action and carry over a fixed quantity of water for each cycle of operation, are by far the most used. Generally, this type should be used on all installations for which the maximum demand does not exceed 160 gpm.

The current meter, which registers flow by recording the revolutions of an impeller, set in motion by the force of the flowing water, has but little use in water distribution. Because of its low accuracy on small flows, its use is restricted to railroad standpipes, elevators, water carts, water motors, irrigation systems and other installations requiring free discharge and heavy service, with little probability of small rates of flow during day or night.

Compound meters combine a main meter of the current or displacement type, for measuring large flows, with a small, displacement type meter, for measuring small flows, in conjunction with an automatic valve mechanism. In the author's judgment, this is the correct instrument for measuring water when the demand varies between 160 gpm. and 1,000 gpm. For larger flows a battery of compound meters should be used. Some operators favor a battery of displacement meters instead of a compound meter because of the low accuracy during the "change-over" from the by-pass to the main unit of the compound meter. The selection of the proper size compound meter to a degree eliminates this objection.

The fire service meter is a compound meter in which a proportional type main unit, for measuring large flows, combines with a small displacement type unit, for measuring small flows, in conjunction with an automatic valve mechanism. This combination is designed to afford an unobstructed passage through the main meter when its valve is raised from the seat. The measurement of flow recorded on the main unit, through which only a relatively small portion of the total flow actually passes, is likely to be inaccurate unless care is taken to keep all parts of the meter clean and in the very best working condition. The fire service meter should never be used for general service.

Selecting Meter Sizes

The selection of the correct type of meter is relatively easy, but the determination of the proper size is complicated and requires careful calculation coupled with experience and practical knowledge. The size depends largely upon the expendable pressure available, and may be determined by learning the maximum demand and permissible pressure loss and by referring to a capacity curve of the meter to be used.

To estimate the maximum demand before service for many establishments, one can refer to any of the standard plumbers' manuals, in which the required capacity for various fixtures is given in gallons per minute and the probability of simultaneous operation is estimated. Table II, taken from a handbook published by the Copper and Brass Research Assn., offers an example of such capacity estimations.

TABLE II

Rates of Flow Required by Plumbing Fixtures

Fixture	Flow*
Bath	8-10
Lavatory	5- 8
Tank closet	3- 5
Flush valve closet	30-40
Shower	5- 8
Sink	8-10
Laundry tub	8-10
Garden hose	5-10

*gpm.

The rate of flow may be approximated, as a rule, by dividing by four the amounts given in Table II for residences, apartments, schools, office buildings and other buildings in which the character of water consumption is similar. For clubs and hotels, the amounts should be divided by three; for gymnasiums, hospitals, public comfort stations and other such institutions, by two; and for public baths, laundries and factories, the full amount should be allowed. Often special indications will require that factors differing from those recommended be used. At best, all these factors are arbitrary.

As an alternative, the method of calculating the maximum demand outlined by the National Bureau of Standards may be used. The essentials of this outline consist of a table of demand weights for different plumbing fixtures and curves from which the estimated demand in gallons per minute corresponding to any total number of fixture units may be obtained. The demand weights, presented in terms of fixture units for different plumbing fixtures under different conditions of service, appear in Table III.

One progressive water department[1] has developed a set of graphs (Figs. 1 and 2) for determining the proper size of service connections and meters. With the number and type of fixtures known, the probable

TABLE III

Demand Weights of Fixtures in Units*

Fixture† or Group	Occupancy	Type of Control	Weight in Fixture Units‡
Water closet	Public	Flush valve	10
Water closet	Public	Flush tank	5
Pedestal urinal	Public	Flush valve	10
Stall or wall urinal	Public	Flush valve	5
Stall or wall urinal	Public	Flush tank	3
Lavatory	Public	Faucet	2
Bathtub	Public	Faucet	4
Shower head	Public	Mixing valve	4
Service sink	Office, etc.	Faucet	3
Kitchen sink	Hotel or restaurant	Faucet	3
Water closet	Private	Flush valve	6
Water closet	Private	Flush tank	3
Lavatory	Private	Faucet	1
Bathtub	Private	Faucet	2
Shower head	Private	Mixing valve	2
Bathroom group	Private	Flush valve for closet	8
Bathroom group	Private	Flush tank for closet	6
Separate shower	Private	Mixing valve	2
Kitchen sink	Private	Faucet	2
Laundry trays (1-3)	Private	Faucet	3
Combination fixture	Private	Faucet	3

*Supply outlets likely to impose continuous demands should be estimated separately and added to the total demand of the fixtures.

†Weights of fixtures not listed may be assumed by comparison with one using water in similar quantities and at similar rates.

‡For total demand. Weights of the maximum separate demands of fixtures with both hot and cold water supplies may be taken to be ¾ of the demand listed for the supply.

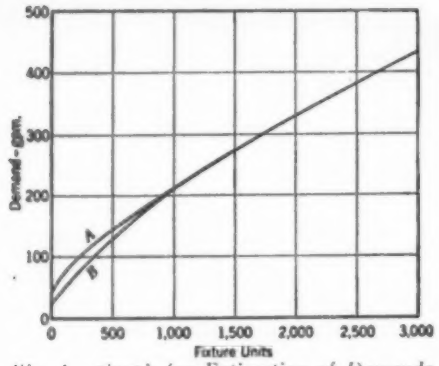


Fig. 1.—Graph for Estimation of Demands (For enlarged section see Fig. 2)

maximum demand may be estimated. The total length of the service pipe may then be ascertained and the proper size of the service connection and meter determined from the graphs.

Any estimation of the demand of manufacturing establishments must include the quantity required by the manufacturing processes. This information can usually be obtained from the customer or the manufacturer of the equipment installed.

Checking Meter Sizes

Whether a meter already in service is of the proper size may be ascertained by attaching a recording mechanism to the meter and obtaining data on the maximum and average rates of demand.

Another very useful method for determining whether the meters in a water system are of the proper size is to establish a load factor for the meters and let the regular meter readings show whether a meter is under- or overworked. A load factor, based on the permissible loss of pressure through the meter, has to be established for each community and sometimes for separate districts within a community, if the pressure variations are large. The load factor is the ratio of the average delivery to the rated delivery. The rated delivery must not be confused with the manufacturer's rating or the safe operating capacity of the meters, as such quantities are based on too great a pressure loss (15-20 psi.).

New York City uses a rated delivery based on a 0.25 psi. loss, and if the load factor is below 30 per cent of the rated delivery shown in Table IV the meter is considered too large; if above 50 per cent, too small. Allen Hazen[2] uses a rated delivery based on a 5 psi. loss and considers the meter to be underworked if the load factor is 2 per cent or less. Table V shows the rated delivery used for the calculation of the load factors.

TABLE IV

Rated Delivery of N. Y. City Meters*

Meter Size in.	Rated Delivery gpm.	Rated Delivery mil. gal. per year
DISPLACEMENT METERS		
⅝	2.0	1.1
¾	3.4	1.8
1	5.3	2.8
1½	10.0	5.3
2	16.0	8.4
3	31.5	16.6
4	50.0	26.3
6	100.0	52.6
CURRENT METERS		
1½	10.0	5.3
2	17.5	9.2
3	40.0	21.0
4	70.0	36.8
6	160.0	84.1
8	280.0	147.2
10	437.5	230.0
12	640.0	336.4

*The load factor of compound meters is determined for each unit separately, and not for the meter as a whole.

TABLE V

Rated Delivery Used by Hazen for Displacement Meters

Meter Size in.	Rated Delivery gpm.	Rated Delivery mil. gal. per year
DISPLACEMENT METERS		
⅝	7.11	3.74
¾	12.09	6.36
1	21.33	11.22
1½	42.67	22.44
2	71.70	37.40
3	142.20	74.81
4	213.30	112.21
6	426.70	224.42

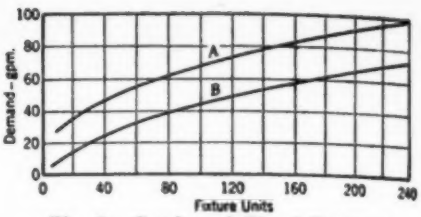


Fig. 2.—Section of Fig. 1 Enlarged Curve "A" applicable to systems in which flushometer valves predominate; curve "B" to systems in which flush tanks predominate.

References

[1]Smith, M. C.: "Standards for Water Service"—**W. W. & S.**, 21:191 (1944).
[2]Hazen, Allen: "Meter Rates for Water Works"—(1918), John Wiley & Sons, New York.

HOW TO PACK A STUFFING BOX

1. Remove all old packing from the stuffing box.
2. Clean box thoroughly, removing accumulations from walls.
3. Keep packing clean during application. The scoring of rods, shafts or sleeves often results from grit or foreign matter picked up by packing.
4. Determine size of packing required by checking diameter of rod or shaft and the inside diameter of the stuffing box. Rods wear and the packing has to compensate for this wear.
5. Never wind a length of packing into a stuffing box. Use ring form only.
6. Cut spiral or coil from packing over a shaft or mandrel of exactly the same size as the one on which the packing is to be used in order to get close fitting rings.
7. Obtain sharp clean cuts on soft packings that tend to squash or become ragged by simply applying a small piece of friction tape at the place where the packing is to be cut. It is advisable to remove the tape when installing the rings. String also serves well for tying the packing on each side of where the cut is to be made and may be left on the packing when installed.
8. Cut all packing ring joints to a snug fit making the rings meet with a tight butt or bevel joint.
9. Seat each packing ring carefully before placing the next one in the box. Loading a stuffing box with packing and then pulling up on the gland follower may cause excessive deformation of the first two rings, building up localized gland pressure and failure to seat the bottom rings so they can work effectively.
10. Install all packing rings so that joints are staggered and kept approximately 90 deg. apart. Spot joints in clockwise direction to facilitate locating them when removing packing.
11. Pull up the gland follower sufficiently tight, after the packing has been installed, to insure final seating of the packing. The nuts should be backed off two or three turns to relieve excessive pressure. Adjust packing to operating conditions.
12. Keep the following in mind for stuffing boxes provided with water, oil or grease lanterns: (a) Make sure that the lantern is so located in the box that as the packing is taken up, the lantern will be advanced to its proper location with respect to the opening in the box. (b) Check to see that the opening into the stuffing box is free from obstructions so that the sealing water or lubrication can flow into the lantern.
13. Allow packing to leak a small amount, if possible, when starting a newly packed pump. Take up the gland slowly as the packing seats.
14. Provide means for lubricating the rod and packing wherever possible.
15. Finally, carefully wrap all extra packings, place them back in their container as a precaution against waste and the possibility of their picking up dirt and other injurious foreign substances.

Courtesy of *Power*

TREATING WATER TO PREVENT CORROSION

By JOHN R. BAYLIS

Editorial Associate, Water & Sewage Works

The writer, in 1930 presented some of the factors involved in preventing corrosion of water pipes.[1] Curves were given to show the solubility equilibrium of iron and zinc compounds in the presence of their respective metals. These curves (Fig. 1) show that practically no iron or zinc will exist in solution when the pH is above 8.0. At a pH of 7.0 several parts per million of either metal may exist in solution in some form in the absence of oxygen, and at a pH of 6.5 quite a large amount of iron or zinc may be found in solution in the absence of oxygen. The article explained how corroding iron took all the free CO_2 and much of the half-bound CO_2 from the water, leaving the water distinctly alkaline where there was no dissolved oxygen present. If the water contained dissolved oxygen the iron was precipitated as a hydrous oxide. Section of a 1-in. cement lined pipe, which had been in service for 50 years at Danvers, Mass., also was shown. The interior of the pipe was in fairly good condition, indicating that the lining was still giving very good protection.

Carrying Capacity of Pipes Greatly Reduced by Incrustation Resulting from Corrosion.—That the carrying capacity of iron pipes may be greatly reduced by tuberculation and incrustation as a result of corrosion is well known to most every one.

Haydock[2] describes how cleaning the tubercles or incrustation from several pipe lines increased the flow. One line carried only about one-half its theoretical capacity, and another about two-thirds. Cleaning a pipe line 3 miles long increased the capacity to the extent that a pumping station could be abandoned, and cleaning another line 5 miles long also enabled the abandonment of a pumping station. The friction loss through a 12-in. line 4 miles long 20 years old was nearly three times that of a new line. The capacity was increased by approximately 35 per cent and the delivered pressure by 20 lb. The gravity delivery of a 16-in. pipe 10.5 miles long was increased 60 per cent by cleaning. Cleaning a 13 mile section of a 12-in. pipe line 22 miles long, which had been in service about 16 years, increased the capacity approximately 35 per cent, and cleaning another section of the line 6 miles in length increased the carrying capacity to 46 per cent.

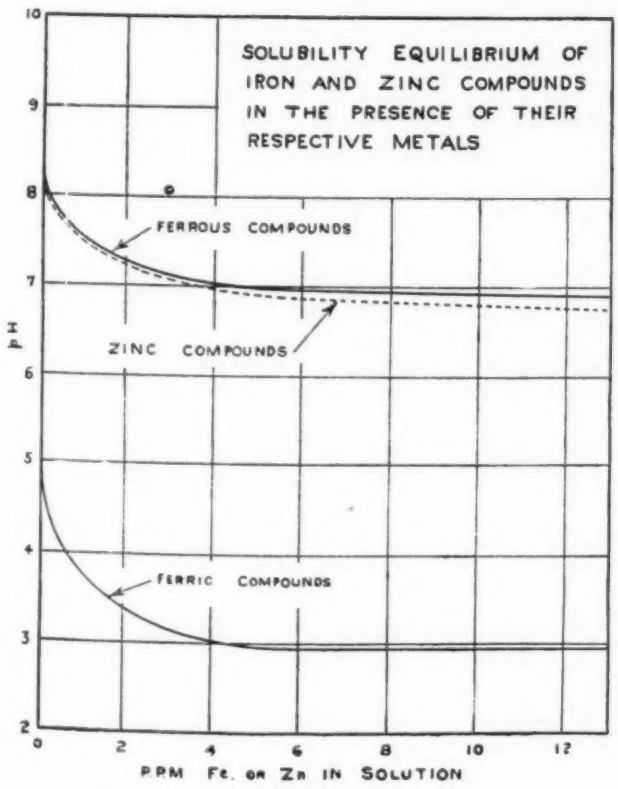

SOLUBILITY EQUILIBRIUM OF IRON AND ZINC COMPOUNDS IN THE PRESENCE OF THEIR RESPECTIVE METALS
FERROUS COMPOUNDS
ZINC COMPOUNDS
FERRIC COMPOUNDS
pH
PPM Fe. or Zn IN SOLUTION

Fig. 1

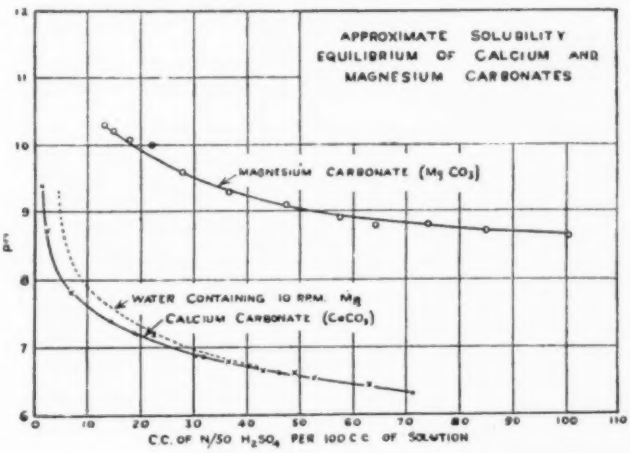

APPROXIMATE SOLUBILITY EQUILIBRIUM OF CALCIUM AND MAGNESIUM CARBONATES
MAGNESIUM CARBONATE ($MgCO_3$)
WATER CONTAINING 10 PPM. Mg
CALCIUM CARBONATE ($CaCO_3$)
pH
CC. OF N/50 H_2SO_4 PER 100 C C OF SOLUTION

Fig. 2

Increased Pumping Pressure or Larger Mains Very Expensive.—In some instances it is possible to deliver the desired amount of water through distribution systems that have become partially clogged by increasing the pumping pressure as the friction losses increase, or to install booster pumping stations on the distribution system in certain sections where the pressure is low. Haydock refers to the delivered pressure being increased 20 lb. by cleaning the pipes in one instance. Assume that it was desired to maintain this 20 lb. increased pressure, evidently it was quite expensive to do it at the pumping station. A fair average of the cost of lifting one million gallons of water 1 ft. in height is probably 5 ct. Twenty pounds is equivalent to about 46 ft. in head. In this case it would cost about $2.30 per million gallons to maintain the increased pressure.

If corrosion is going to reduce the carrying capacity of the water mains in a certain city 50 per cent within 20 years, what is the best way of handling such a situation? Should mains be installed large enough to give the desired capacity with a 50 per cent reduction in carrying capacity? The cost of such an investment over what would be necessary if there was no corrosion very likely would be much more than the cost of making the water reasonably non-corrosive.

With the exception of feeder mains, engineers usually design the water distribution system for fire protection. How many engineers allow for a 50 per cent reduction in the carrying capacity of the pipes within 20 years where the water is corrosive? How many make any distinction whatever in the design where the water is corrosive and where it is not corrosive? The writer is of the opinion that corrosion has not been given its proper consideration in the design of water systems. This is largely due to the lack of specific information in the past as to how much the carrying capacity will be reduced within a certain time.

Treatment with an Alkali for Partial Protection.—The literature on the treatment of water to prevent corrosion indicates quite conclusively that the corrosiveness of water may be greatly reduced by proper treatment. It seems that some of the authors of articles on corrosion prevention do not fully realize the importance of a film for protection. So long as iron stains are prevented, some think that this is all the treatment that is necessary. As a matter of fact the carrying capacity of the pipes may be reduced just as rapidly as before, because most of the corroding iron is deposited near where the corrosion is taking place and forms large tubercles that are more effective in reducing the carrying capacity of the pipes than if the rust was

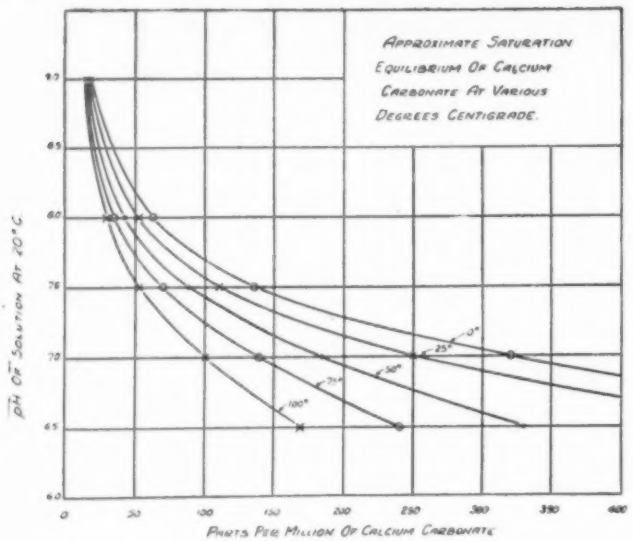


Fig. 3

deposited fairly uniform over the surface of the pipe. Treatment just to the point where red water complaints cease may not materially reduce the cost to the water department for cleaning pipes, constructing new mains, and increasing the pumping pressure.

Water Just Saturated with Calcium Carbonate is the Most Desirable Equilibrium.—As a general rule, it might be said that water saturated with calcium carbonate is the most desirable equilibrium when everything is taken into consideration. This is not strictly true for very soft waters, for it is desirable to have the water slightly supersaturated part of the time so as to form a thin protective film. Where the calcium carbonate alkalinity of the water is below 30 parts per million the application of lime to produce a slight supersaturation of calcium carbonate should be done with considerable care, otherwise caustic water might be produced. After a thin coating has been formed on the pipes the water should be kept saturated with calcium carbonate practically all the time to keep the coating from being dissolved. It is admitted that the very soft waters are the most difficult to protect, for we are working with equilibriums difficult to maintain exactly on the saturation point. It may be almost impossible to do this with the force available in small plants and it may be necessary to sacrifice some protection to avoid occasional caustic taste in the water. There is no reason why large plants with skilled supervision should not be able to maintain a fairly constant equilibrium.

Table I—Rapid Increase in pH Caused by Corroding Iron

Alkalinity	Time in Contact with the Iron Minutes	pH	Fe in Solution
39	0	7.2	
	1	7.3	
	2	7.4	
	3	7.7	
	5	8.2	
	10	9.2	
	15	9.4	
35	20	9.6	0.0tr

Where water has a calcium carbonate alkalinity in excess of about 30 it is easier to protect the pipes. If this water is placed in contact with an iron surface the iron going into solution will take up all the free Co_3 that is in the water and part of the half-bound CO_3. Table I shows how quickly the pH changes in water exposed to iron lathe turnings. After the pH reaches 8.1 no free CO_3 is present. This was due to the CO_3 combining with the corroding iron.

At any point on the surface of an iron pipe where corrosion starts to take place due to the surface becoming exposed in some manner, there is a tendency for the iron going into solution to take up the free and part of the half-bound CO_3, producing a supersaturation of calcium carbonate in the immediate point where corrosion is taking place. This supersaturation helps to quickly form a coating over the exposed surface both by precipitating the iron compounds and also some calcium carbonate, especially when the water has a fairly high alkalinity. If the water in the mains is saturated with calcium carbonate, then the precipitated calcium carbonate will not be dissolved out of the percipitate. It will remain where it is precipitated to offer protection against further corrosion.

Most natural waters contain magnesium carbonate as well as calcium, and the presence of a considerable amount of magnesium may materially alter the equilibrium curve. This is shown by the curves in Fig. 2. The presence of sodium carbonate also alters the curve. When sodium carbonate is present the only way the saturation equilibrium of the water in regards to calcium carbonate can be determined is by actual trial.

Temperature also has an effect upon the saturation equilibrium of calcium carbonate. The curves shown in Fig. 2 were determined at room temperature, that is, about 20 to 22° C. The other curves are based on actual tests, but not enough were run to state that the curves are absolutely correct. They should be regarded as somewhere near the true lines.

Corrosion prevention by treatment of the water has been demonstrated to be practical and economical. This being true, then it seems that all corrosive waters should be treated.

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[1]Baylis, J. R. How to Avoid Loss by Pipe Corrosion. Water Works Engineering, 83:13-14, 31-32, 35-36, Jan. 1, 1930.

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The Influence of Dissolved Oxygen on Corrosion Losses

Engineers and chemists are constantly studying the theories and mechanics of corrosion because it is only through better understanding of fundamentals that they will be able to diminish losses due to corrosion.

Thirty years ago, R. W. Whitney, now vice-president of the general Electric Company, announced that the corrosion of metals was an electrochemical process and this view is now generally accepted. When a metal is in contact with even slightly acidic solutions, corrosion takes place by the chemical process of the metal replacing the hydrogen in the acid. The acid dissolves the metal and thereby hydrogen is released. The same holds for corrosive waters.

Released hydrogen coats the metal surface with an electrically charged layer that prevents the chemical reaction from continuing at the initial rate. If the hydrogen layer is built up to a sufficient thickness the reaction is practically stopped. However, under natural conditions, the hydrogen combines with the oxygen of the air or that dissolved in the water to form water and solution of the metal goes on. The *amount of oxygen* available to remove the protective hydrogen blanket determines the extent of corrosion.

Impure metals are corroded more quickly as a rule than pure metals. Electrochemists believe that this is due to the impurities taking up the oxygen. Contrary to the popular belief, however, pure metals are subject to corrosion under the action of certain acid solutions. Prof. O. P. Watts of the University of Wisconsin has recently shown that if oxygen is absolutely excluded copper will not corrode in a solution of sulfuric acid, a reaction that is very rapid in the open.

The importance of the dissolved oxygen content of corrosive water is thereby once more emphasized. When this element is present in high concentration a higher pH value must be maintained to prevent corrosion of mains than otherwise; and, particularly of household services or piping subjected to increased temperatures which create accelerated attack.

This suggests that there is some question as to the merits of aeration of waters which are normally aggressive, unless the pH value is raised sufficiently by alkali addition to reduce the hydrogen ion concentration considerably; below that which is commonly considered to represent a neutral water. It has in fact been demonstrated that ofttimes waters high in oxygen content and low in hardness must be raised to pH 9.5 in order to check even the cold water corrosion of mains. In some aerated waters a pH value of 10 to 10.5 is required, which is costly and questionable practice with public supplies.

STABILIZATION OF LIME-SOFTENED WATER

A Comparison of Three Methods, and Practical Methods of Testing

By H. O. HARTUNG, *Production Engineer*
St. Louis County Water Co., University City, Missouri

A LIME softened water is usually highly incrustant unless given further treatment to produce non-incrusting properties. The incrusting substance is usually supersaturated calcium carbonate produced in the softening reactions and which did not precipitate in the settling basins. Incrustation occurs whenever such water, in striking the sand grains of a filter or the walls of transmission mains or pipe fittings, unloads some or all of its calcium carbonate in excess of its normal solubility.

Obviously the most desirable way to produce a non-incrusting water is to hold the water in the softening plant until complete precipitation of all supersaturated salts has been effected. In addition, this gives the maximum hardness reduction. Usually it is not possible to accomplish such results in existing softening plants.

Carbonate Adjustment

When it is not possible to completely precipitate the products of the softening reaction, equilibrium must be restored by additional treatment. If it is not restored, incrustation of the filter sand and scaling of the distribution system is the result. The commonly accepted method by which stabilization has been obtained is to convert the supersaturated calcium carbonate, with an acid or acid salt, into a more highly soluble salt of calcium. When carbon dioxide (recarbonation) is used, the supersaturated calcium carbonate is converted into non-precipitating calcium bicarbonate. When alum is used, considerable of the surplus calcium carbonate is converted into a more soluble calcium sulphate. Phosphoric acid, sulphuric acid and nitre cake have also been used in a few instances to convert supersaturated calcium carbonate in the water. The addition of any of these substances to the water to produce equilibrium is accompanied by a lowering of its pH value.

Another Method

Another method of preventing calcium carbonate precipation has attracted considerable interest. The method has been reported successful in several industrial water treatment plants and at one or two municipal water plants. It is simply accomplished by adding one or two parts per million of sodium hexametaphosphate $(NaPO_3)_x$ to the water prior to, or following, filtration. This small amount of sodium hexametaphosphate, necessary to produce stabilization, is not dependent upon the alkalinity or pH of the water to be treated, and does not change the pH of the treated water. Also, the amount necessary for stabilization is only a small fraction of that required to react quantitatively with the calcium in solution. The sodium hexametaphosphate merely inhibits calcium carbonate incrustation. Certain surface-active properties of the molecule of this salt (phosphate glass) apparently explains the phenomena.

The object of this paper is to very briefly compare these two methods of stabilization as applied to lime softened water at the St. Louis County Water Company.

Experiences With the Two Methods

Stabilization of lime softened water at the St. Louis County Water Company filter plant is accomplished, at the present time, by recarbonation. The stack gases from an oil fired furnace are scrubbed, dried and then pumped by a reciprocating compressor into the water. Regulation is obtained by metering and controlling the amount of oil burned. Thus the amount of fuel oil burned each hour depends upon the demand of the water for carbon dioxide and the amount of water pumped during that hour. (Fig. 1 is a schematic sketch of the recarbonation plant.)

The plant is sufficiently large to recarbonate about 30 mgd. of water, depending, of course, upon the water requirements for carbon dioxide. The oil burners can be varied to burn from 3 to 12 gallons of fuel oil per hour. All of the gases from the furnace are drawn into a water spray contact scrubber by the suction created at the compressor. The scrubber is 60 inches in diameter and 13 feet high. The gases rise vertically through 4½ feet of 2-inch gravel and then through 4½ feet of packed excelsior, which is kept wet by overhead water sprays. These sprays use about 1,000 gallons of water per hour. The gases deposit most of the unburned material which they carry on the wet surfaces in the scrubber. From the scrubber the gases pass to a drier, 20 inches in diameter and 15½ feet high, filled with iron scrap and turnings. The fuel oil contains a small percentage of sulphur, and the acids formed by this sulphur are expended upon the exposed iron surfaces in this tower. The gases next pass to an adjacent drier of the same dimensions, but filled with mineral wool and packed excelsior. These materials mechanically entrain much of the remaining moisture, thus protecting the compressor against corrosion to a considerable extent. The compressor then pumps the gases to the water being treated. Stainless steel compressor valves have also minimized corrosion and maintenance of the carbonating equipment.

The operating data obtained during 2 years and 9 months' operation of the recarbonation plant are given in Table I.

Carbonating Efficiencies

The average chemical efficiency of the plant has varied between approximately 55 and 60 per cent. The chemical efficiency of the plant is the ratio of the carbon dioxide utilized to that potentially available in the fuel oil burned. It thus takes into consideration combustion, scrubbing, absorption and grid losses. The greater part of the 40 per cent losses are encountered in the grid system. Much of the carbon dioxide bubbles through the water and escapes to atmosphere. About 10 per cent of the carbon dioxide is lost in the scrubber. Taking into consideration these losses, only about 13 pounds of carbon dioxide from every gallon of fuel oil burned is actually utilized in the process.

The gases from the fuel oil combustion have averaged about 9 per cent carbon dioxide. This percentage should not be confused with the per cent carbon dioxide obtained in a regular fuel oil furnace. In the recarbonation plant the draft on the furnace is kept constant by the suction from the compressor. The fuel oil burned, however, is varied to meet the recarbonation demands of the water. The per cent carbon dioxide thus varies with the amount of fuel burned.

The average of 3.6 parts per million of carbonates converted to bicarbonates, per gallon of oil burned for each million gallons of water treated (see Table I), is a convenient unit figure for specifying the correct oil feed. For example, if the normal carbonates in the water equal 46 ppm. before recarbonation and are to equal 10 ppm. after recarbonation (i.e., the reduction in normal carbonates is to equal 36 ppm.), the oil to be burned per million gallons must equal 36 divided by 3.6, or 10 gallons. The unit figure is an average obtained from actual plant operating data.

Laboratory Control Procedure

The amount of fuel oil to be burned is specified by the control laboratory. The laboratory arrives at the correct result

TABLE I

Recarbonation Plant Report—St. Louis County Water Company

		1936	1937	9 Months, 1938	Av. or Total
Phenolphthalein Alkalinity	Before Recarb.	30	29	26	29
	After Recarb.	9	5	11	9
pH	Before Recarb.	10.3	10.4	10.1	10.2
	After Recarb.	8.9	8.7	9.2	8.9
	Hours Operation	6,387	5,727	5,769	30,464
Water Used in Plant (1000 Gal.)	Boiler	738	697	319	.....
	Scrubber	7,708	6,572	6,489	.....
	Compressor	303	103	128	.....
Fuel Oil Burned Gallons	Total	54,083	49,242	32,378	233,140
	Per M.G.	11.7	12.7	9.0	11.3
	P.P.M. CO_2 to HCO_3 per Gal. Oil per M.G. Water (p.p.m. $CaCO_3$)	3.53	3.78	3.55	3.61
	Estimated Lbs. CO_2 Utilized Per Gal. Oil Burned	13.0	13.9	13.0	13.2
	Overall Chemical Efficiency of Plant (per cent)*	56.6	59.5	55.3	56.6
	% Carbon Dioxide in Scrubbed Gas	10.6	10.9	6.0	9.0

*Chem. Eff. $= \frac{2(P' - P'')}{6.36 \text{ (Gal. Oil Per M.G.)}}$

P' = Phenol. Alk. before recarb. — P'' = Phenol. Alk. after recarb.

TABLE II

Sodium Hexametaphosphate Tests on Lime Softened Water

Initial Water pH	Initial Water Total Alkalinity ppm.	Initial Water after 24 hrs. Contact with $CaCO_3$ Total Alkalinity ppm.	Initial Water Plus Sodium Hexametaphosphate after 24 hrs. Contact with Calcium Carbonate $(NaPO_3)_x$ Dose ppm.	Initial Water Plus Sodium Hexametaphosphate after 24 hrs. Contact with Calcium Carbonate Total Alkalinity ppm.	No. of Daily Tests Making Average
10.5	49	25	2	46	2
10.2	46	25	2	46	7
9.8	50	33	2	49	5
10.1	41	23	1	41	3
9.9	54	28	1	52	6
10.8	61	33	0.5	60	1
10.5	55	24	0.5	55	1
10.1	46	25	0.5	45	16
9.9	47	29	0.5	46	9

TABLE III

Effect of Temperature on Lime Softened Water Stabilized with One Ppm. Sodium Hexametaphosphate

Temperature F°	Original Water pH	Original Water Total Alkalinity	Total Alkalinity After Stability Test Sodium Hexametaphosphate Treated Water	Total Alkalinity After Stability Test Untreated Water	Time Contact with $CaCO_3$ at Temperature Indicated -Hours-	Number of Determinations Different Days
40	10.4	54	53	35	6	3
55	10.4	54	54	34	6	3
75	10.1	42	42	26	6	10
100	10.1	42	42	30	6	10
140	10.1	42	41	31	6	10
180	10.1	42	41	34	6	10
212	10.1	42	38	31	2	10
250	10.1	42	31	29	½	10

from daily calcium carbonate stability tests. About 50 grams of calcium carbonate are added to 500 ml. of water treated with carbon dioxide and then rolled for eight hours on the rolling machine, pictured in Fig. 2. The water is then centrifuged and an alkalinity determination made. A drop in alkalinity or pH after rolling is indicative of an incrustant water. Similarly, an increase in alkalinity or pH after rolling is indicative of overtreatment with carbon dioxide. The plant carbon dioxide dose is adjusted accordingly.

Fig. 3 illustrates a convenient method for determining the pH at which a carbon dioxide treated water is stable. Several samples of lime softened water are treated with various amounts of carbon dioxide. This is done by saturating a sample of the same water with carbon dioxide. Then, with a pipette, various quantities of the carbonic acid solution thus formed are applied to the water under test. Next the pH of each sample is carefully measured and recorded. After adding calcium carbonate, the samples are rolled for about eight hours, centrifuged and the change in alkalinity noted. By plotting the change in alkalinity against the pH of the water before the addition of calcium carbonate, the pH at which there is no change in alkalinity can be read. This is the stability pH which should be obtained in the plant.

Hexametaphosphate Treatment for Stability

Calcium carbonate precipitation may also be prevented by the addition of 1 to 2 ppm. sodium hexametaphosphate ($Na PO_3)_x$. This effect is produced regardless of the amount of calcium in the water[1] and without a change in pH. In hot water systems somewhat larger dosages have been reported to effectively disintegrate old scale deposits.

The effectiveness of 1 or 2 ppm. of sodium hexametaphosphate in preventing calcium carbonate precipitation may be conveniently demonstrated in the following experiment. To 500 ml. of water containing from 50 to 150 ppm. calcium bicarbonate add 2 ppm. of sodium hexametaphosphate and 250 ppm. of soda ash. To a duplicate sample add 250 ppm. of soda ash.

Precipitation in the second sample will usually occur within five minutes, while in the sample containing the hexametaphosphate the reaction will be delayed many hours to several days.[2]

Table II gives the results of calcium carbonate stability tests on lime softened water, taken from the basins of the St. Louis County Water Company, with and without treatment with sodium hexametaphosphate. This table shows that high pH waters were stable after 24 hours when treated with as little as 0.5 ppm. of sodium hexametaphosphate, as determined by the changes in alkalinities. Samples treated with sodium hexametaphosphate had very little, if any, change in alkalinity after contact with calcium carbonate.

Each day, for a period of about two months, two 250 ml. water samples were taken from the softening basins. Sodium hexametaphosphate was added to one sample in the amounts indicated in the table. One gram of pulverant calcium carbonate was then added to both samples. The water samples completely filled the sample bottles, which were tightly stoppered. Both sample bottles were then rolled for eight hours in the machine (Fig. 2), and the change in alkalinity (if any) determined after 24 hours. The same calcium carbonate was used in each succeeding day's test. Table II reveals the average of these determinations.

Sodium hexametaphosphate will revert to the orthophosphate form upon prolonged heating and thus lose its surface-active properties. (Tri-sodium phosphate is an orthophosphate.) Table III gives the stabilization effect of sodium hexametaphosphate on lime softened waters heated

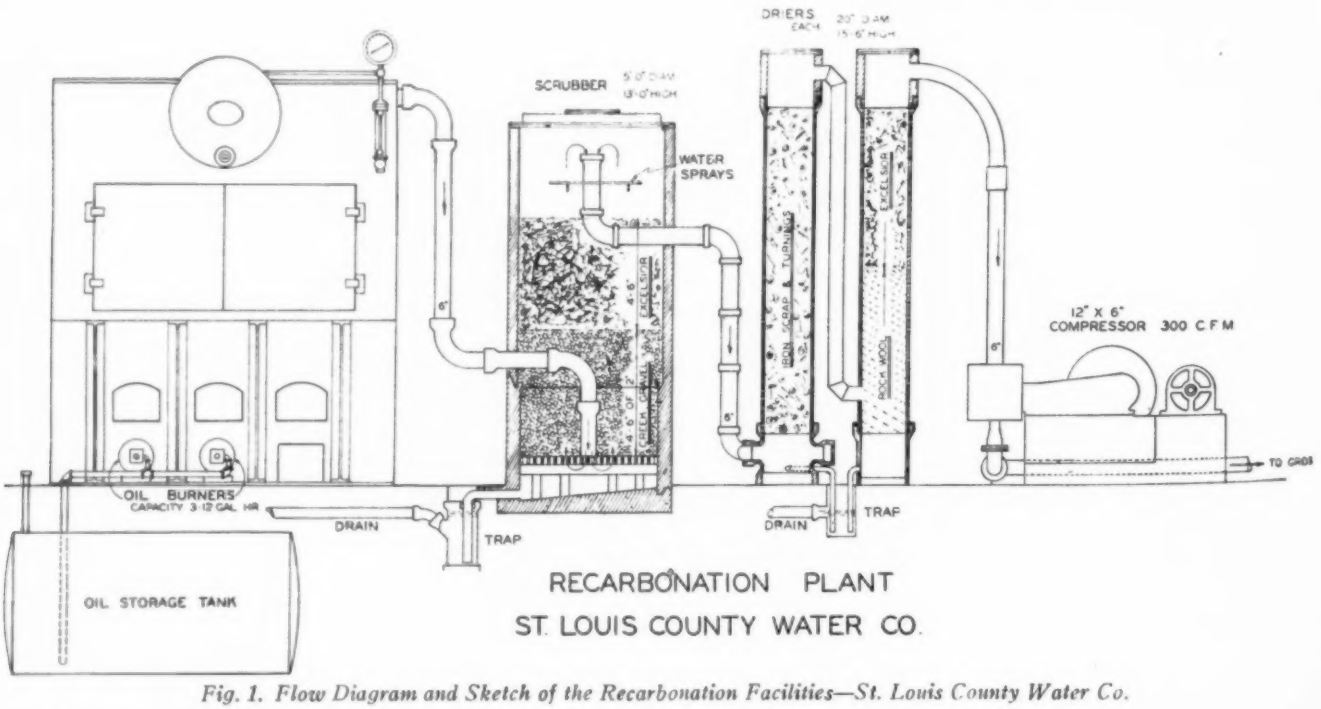

Fig. 1. Flow Diagram and Sketch of the Recarbonation Facilities—St. Louis County Water Co.

to temperatures encountered in the ordinary domestic hot water system. According to these laboratory results, one ppm. of sodium hexametaphosphate effectively stabilizes the water up to about 200° F. However, at boiling temperatures it is not so effective. But even at these temperatures as little as one ppm. of hexametaphosphate has some effect in pegging the alkalinity. At 250° F. the stabilizing effect of such a low dosage of sodium hexametaphosphate is negligible from a practical point of view.

The stability tests for water heated up to 200° F. were made as before but in vacuum ("Thermos") bottles. About 0.5 gram of calcium carbonate was added to 500 ml. of water heated to the proper temperature and placed in vacuum bottles, and then rolled for 6 hours on the rolling machine. After 6 hours the water was cooled, centrifuged and alkalinity again determined. Fresh calcium carbonate was used in each test.

Tests at 212° F. were made by almost completely submerging the test bottles fitted with loose stoppers in a water bath for two hours. The tests at 250° F. were made in a steam autoclave for 30 minutes.

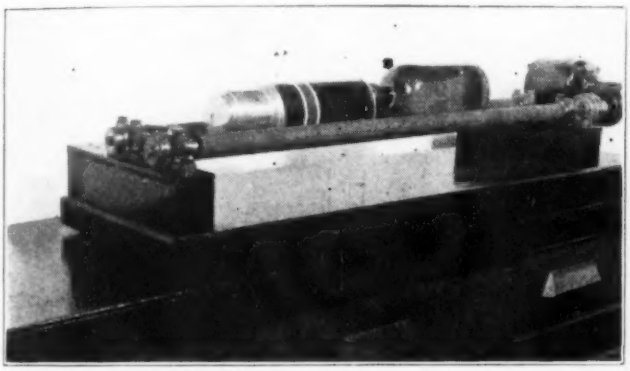

Fig. 2. The Rolling Machine for Making the Routine Marble Test, Eight Hours Being Required to Attain Equilibrium with St. Louis County Water.

The alkalinities reported for these two temperatures were adjusted to compensate for volume changes resulting from heating and cooling.

Water which is stabilized by recarbonation at room temperature often loses its stable characteristics at higher temperatures. Table IV shows that seemingly nothing is gained by adding sodium hexametaphosphate to water which has been recarbonated to stability and then heated to boiling temperatures. That is, sodium hexametaphosphate did not prevent calcium bicarbonate in carbonated water from breaking down into calcium carbonate, such as occurs in hot water systems.

Hexametaphosphate vs. Recarbonation

Sodium hexametaphosphate may be easily fed as a solution. Since it is noncorrosive, any type of solution feeder may be used. A simple siphon feeder from a barrel or drum is satisfactory.

Sodium hexametaphosphate is an effective stabilization agent in concentrations from about 0.5 to 10 ppm. Consequently, close regulation of feed is not necessary. Periodic check on the feeder is all that would be required.

About 0.75 ppm. sodium hexametaphosphate, at present prices, is approximately equivalent in cost to recarbonation at St. Louis County Water Company. Sodium hexametaphosphate treatment seems to have an advantage over recarbonation in that stability is much easier to obtain, and the simpler equipment requires less investment, less space, and is much less costly to maintain.

The recarbonation plant at the St. Louis County Water Company requires a monthly maintenance program which necessitates the plant to be out of service for at least one day while the compressor is being cleaned or the materials in the drier are being changed. No such out of service period or labor and materials cost, would be necessary for the sodium hexametaphosphate feed because duplicate feeders may be installed at small cost.

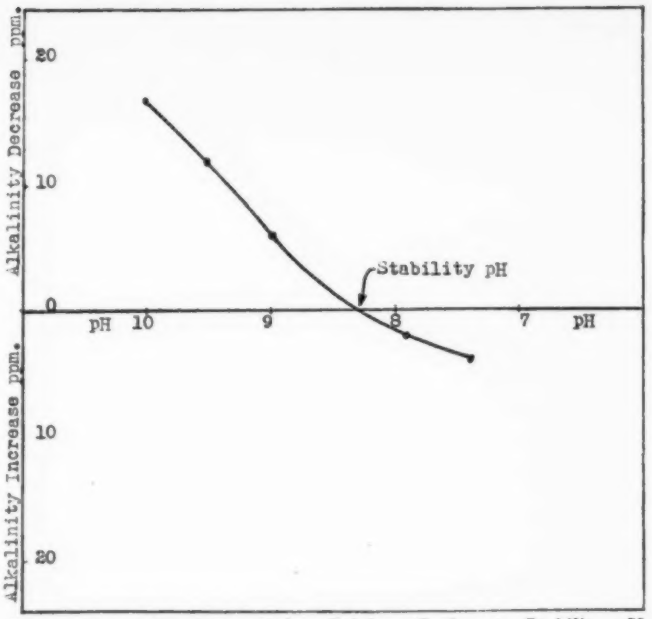

Fig. 3. Graph for Determining Calcium Carbonate Stability pH. (Missouri River Lime Softened Water—Oct., 1938.)

Stabilization with Tetra-Sodium Pyrophosphate

In order to determine whether certain other phosphate salts might have surface-active properties similar to those of sodium hexametaphosphate, some of the previous stability tests were repeated, using tetra-sodium pyrophosphate instead of sodium hexametaphosphate. Table V shows that as little as 0.05 ppm. of tetra-sodium pyrophosphate was also effective in stabilizing the water at room temperature for at least 24 hours. At 212° F. tetra-sodium pyrophosphate was less effective in preventing incrustation than at room temperature. With a treatment of 2 ppm. there was some calcium carbonate deposition within 2 hours at the boiling temperature.

Table VI shows that 1 ppm. of tetra-sodium pyrophosphate was an effective stabilization agent until the water approached boiling temperature. At 250° F. the tetra-sodium pyrophosphate had very little effect in holding the alkalinity at its original value. All of these results may be compared with results obtained when stabilization was obtained with sodium hexametaphosphate (see Table III) and there appears little if anything to choose between the two salts.

Qualitative tests made in the laboratory, however, seem to indicate that water with high calcium concentrations are not so easily (if at all) stabilized with tetra-sodium pyrophosphate. This seems to be at least one differentiating characteristic between sodium hexametaphosphate and tetra-sodium pyrophosphate for stabilization.

Conclusions

The present accepted and proven method for transforming a lime softened water into a stable water is by recarbonation. However, new methods are constantly being developed which merit the attention and study of the water works operator. Sodium hexametaphosphate "Threshold Treatment" seems to be one of these. The

TABLE IV

Effect of Sodium Hexametaphosphate Added to Water Already Stabilized by Recarbonation

Temp. F°	Water Stabilized by Recarbonation at Room Temperature — Alkalinity P	Alkalinity M.O.	pH	Chemical Characteristics — Recarbonated Water—Untreated — Alkalinity P	Alkalinity M.O.	pH	After Stability Tests Recarbonated Water Treated with Hexametaphosphate — Alkalinity P	Alkalinity M.O.	pH
212	2	34	8.4	7	35	9.0	8	35	9.0
250	2	34	8.4	11	34	9.2	10	35	9.2

(Results given are the average of five determinations made on different days.)

TABLE V

Tetra-Sodium Pyrophosphate Stability Tests on Lime Softened Water

Original Water — Alkalinity P	Alkalinity M.Or.	pH	$Na_4P_2O_7$ Dose ppm.	Total Alkalinity After Stability Tests 24 Hrs. @ 75° F	2 Hrs. @ 212° F
23	46	10.1	0	26	33
....	....	....	0.5	46	42
....	....	....	1.0	46	41
....	....	....	2.0	48	42

(The above results are an average of 5 determinations made on different days.)

TABLE VI

Effect of Temperature on Lime Softened Water Stabilized with One ppm. Tetra Sodium Pyrophosphate

Temperature F°	Original Water pH	Original Water Total Alkalinity	Total Alkalinity After Stability Test $Na_4P_2O_7$ Treated Water	Total Alkalinity After Stability Test Untreated Water	Time Contact with $CaCO_3$ at Temperature Indicated -Hours-
40	10.4	54	53	35	6
55	10.4	54	54	34	6
75	10.0	44	46	28	6
100	10.0	44	47	29	6
140	10.0	44	44	32	6
180	10.0	44	43	37	6
212	10.0	44	41	33	2
250	10.0	44	35	30	½

(The above results are an average of 3 determinations made on different days.)

cooperative study of the water works profession will prove whether it is universally adaptable in public water supply treatment. Other phosphate salts, such as sodium pyrophosphate, may also accomplish the same results and suggests further research.

References

[1]"Threshold Treatment" by Chas. P. Hoover, Chemical Engr., Columbus, Ohio, and Owen Rice, Hall Laboratories, Pittsburgh, Pa., in *Water Works and Sewerage*, January, 1939.
[2]Suggested by Hall Laboratories, Pittsburgh, Pa., and confirmed by author.

A CONTROLLER FOR FEEDING MILK OF LIME

A. A. BAILEY
Filtration Engineer, Baltimore, Md.

UNLESS constantly watched, lime solution is hard to control accurately due to clogging of orifice openings, especially at low flows. The best arrangement we have found for handling milk of lime is a simple pinch valve made of a piece of rubber hose, preferably sand blast hose. This type of hose is soft but has enough rigidity to hold its shape even under suction.

At the present time we have two of these devices in use. In one, shown in Fig. 1, control is accomplished by a cam pressing against a swinging plate, which in turn flattens the hose to secure the desired opening. The arm which operates the cam clamps to a quadrant to maintain the desired setting. The quadrant is graduated to show the degrees of opening.

In the second device, shown in Fig. 2, control is accomplished by using an old 2½-inch valve bonnet and stem, welded to the framework which supports the hose. The bar that presses against the hose is fastened to the inner end of the stem so that the stem is free to turn. This system gives a better control than the first because a finer adjustment is possible than with the cam and lever.

Any clogging which may occur in the restricted section of the hose may be easily and quickly eliminated by opening the valve wide for a second and returning it to its original position. Lime does not have the same tendency to build up in the hose as it does on metal parts.

We have found this arrangement to give control as accurate as obtained with metal valves but with less attention. The only critical material required is about 10 inches of rubber hose.

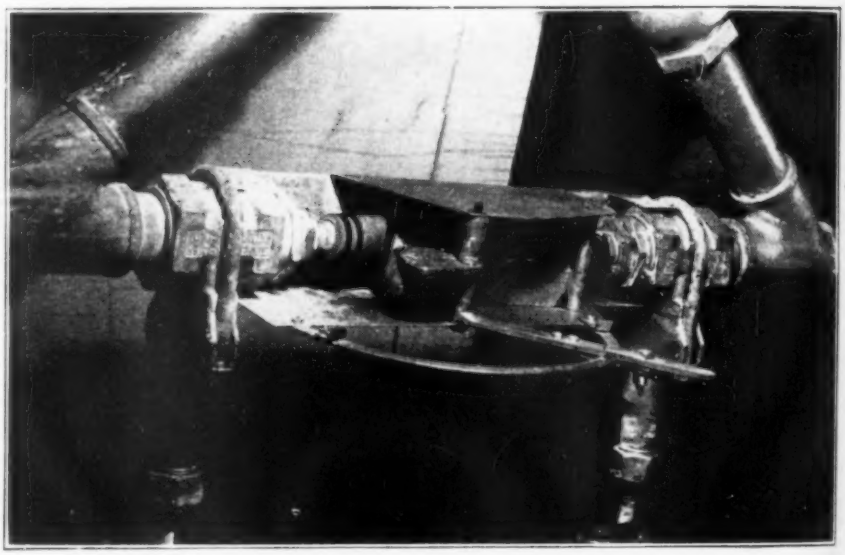

Fig. 1—Lime Solution Feed Control Using Cam and Swinging Plate

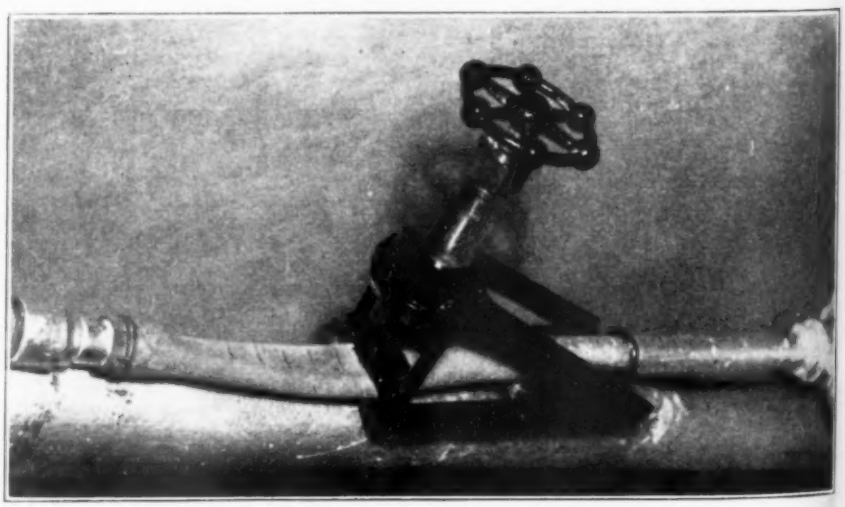

Fig. 2—Lime Solution Feed Control Using Valve Stem and Plate

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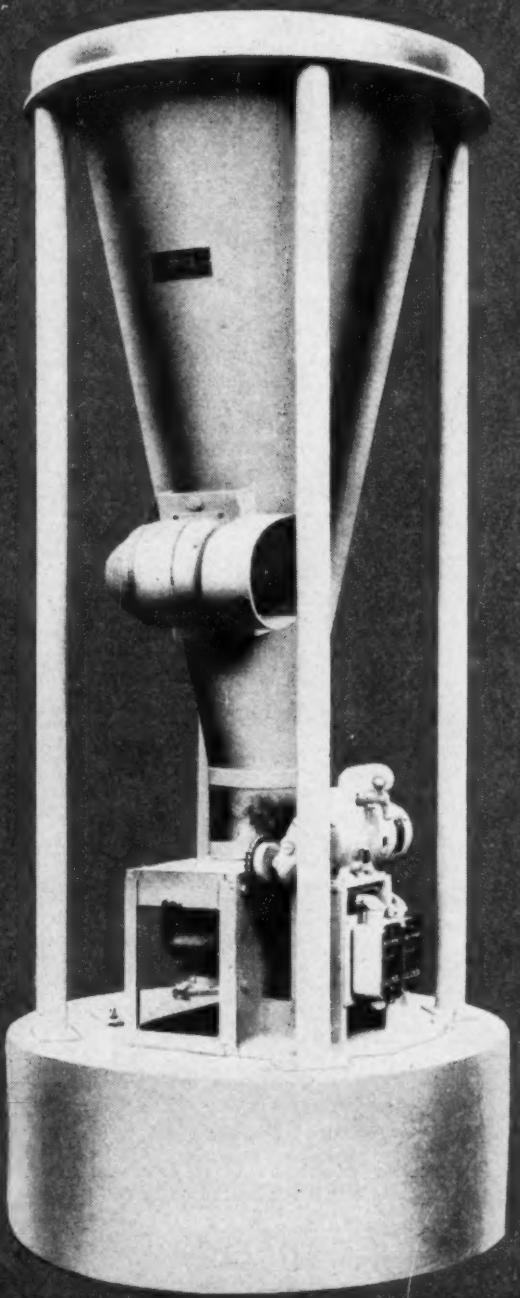

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CALGON TREATMENT*

As Applied in Water Quality Control for the Solution of Four Problems

By OWEN RICE
Calgon Inc., Pittsburgh, Pa.

CALGON is a molecularly dehydrated, glassy phosphate commonly known as hexameta-phosphate and usually given the formula $(NaPO_3)_6$. It is easily dissolved and fed by any solution feed device. For the past ten years it has been of interest and of increasing importance in many phases of water treatment. Its use in several hundred plants during the last six years has shown that the continuous application of small amounts of Calgon to municipal water systems overcomes many of the problems commonly encountered.

Calgon prevents the precipitation of calcium carbonate from lime or lime-soda softened water as well as from hard waters high in bicarbonate alkalinity. Its addition to well water prevents the precipitation of dissolved iron therefrom, while its addition to any water reduces corrosion throughout the system. At first glance, these various applications and effects do not seem related but the result obtained in each case appears to be due to the adsorption of the Calgon on the surfaces of the metals, their metallic salts and oxides involved.

Stabilization of Water After Lime or Lime-Soda Treatment

After-precipitation of calcium carbonate from lime or lime-soda softened water has been a problem ever since the first softening plants were placed in operation. Although after-precipitation can be prevented more or less completely by recarbonation, the limits between which such a treatment must be maintained are very narrow and, although scale may be prevented in this manner, corrosion often results particularly in hot water heaters.

The addition of 1 or 2 ppm. of Calgon completely prevents after-precipitation from lime or lime-soda softened water.[1] It is believed that the metaphosphate is adsorbed on the crystal nuclei of calcium carbonate as rapidly as they form in the water, thus isolating each nuclei from further contact with the water and preventing its growth. This property of Calgon brings about the stabilization of a supersaturated water, from which calcium carbonate would otherwise precipitate.

The addition of Calgon to the water passing to the filters will keep the sand and underdrainage system free from incrustation. In cases where the additional softening effect obtained by the removal of some of the hardness on the filters is desired, the metaphosphate should be added to the clear well. Stabilization with Calgon will result in mains, meters, and hot water systems remaining free from calcium carbonate scale.

Precipitation of $CaCO_3$ from Hard, High Bicarbonate Waters

Calgon has the same stabilizing effect on waters which are naturally hard and high in bicarbonate alkalinity. Where ordinarily the loss of CO_2 or an increase in temperature would cause the formation of scale, this deposition is prevented by the presence of small amounts of Calgon.

Calgon treatment has proven entirely successful in preventing scale formation in hot water heaters with waters up to 1400 ppm. hardness and 600 ppm. bicarbonate, and at temperatures up to the atmospheric boiling point. At the most, an addition of 3 ppm. is required while, under less severe conditions, 2 or even 1 ppm. has proved sufficient.[2]

Preventing Dissolved Iron Precipitation

The action of Calgon on iron in solution is akin to that with calcium carbonate. Precipitation is prevented, thus eliminating "red water." In order to obtain this action, it is necessary to add the Calgon before the iron has had any chance to come into contact with the air. Therefore, in the case of well water supplies it is always best to add the Calgon at the suction side of the pump.

The addition of Calgon in a ratio of 2 ppm. of Calgon for each ppm. of dissolved iron is sufficient to hold the iron in solution even when the water is later aerated or chlorinated.[3]

Corrosion Control with Metaphosphate

Metallic iron when exposed to water dissolves to a considerable extent. Any method of corrosion control depends, therefore, on the formation of an insoluble coating on the surface of the metal.

As mentioned above, Calgon has the ability to coat out on metal or metal oxide surfaces. As the metaphosphate is adsorbed on the iron surfaces of the mains in a distribution system, it forms an iron phosphate film which effectively seals the metal from the water. This film is formed at all pH values above 5.0 so that when using Calgon it is not necessary to add any additional chemicals to change the pH. Also since the film is formed by adsorption and not precipitation, the coating is formed throughout the entire system and not just near the plant.

The addition of Calgon in decreasing corrosion is in no way due to reaction with the water. The water simply acts as a carrier to bring the Calgon in contact with the metal or oxide surface upon which it is adsorbed. Since it is the adsorption of the metaphosphate on the surface of the metal which prevents corrosion, the necessity of supplying sufficient Calgon to meet the requirements of the system is obvious. This can be done either by using a relatively high concentration of Calgon in a small quantity of water or lower concentrations in larger quantities of water. This latter condition is the one which exists in all but the smallest municipal water systems.

In cases where corrosion is severe, it has been found that high initial feeds of Calgon—5 to 10 ppm.—are very helpful in more promptly forming the protective coating in the system. After equilibrium has been reached and the protective coating has been formed, the Calgon can usually be reduced to very small amounts, 1 to 2 ppm., and still be sufficient to maintain the protective coating initially formed.[4]

Summary

Calgon is easily dissolved and fed by any liquid feeding device. It is inexpensive because of the small quantities required and is easy to control. It has proven entirely successful when used in each of the above mentioned applications. Excellent results have been obtained in preventing the after-precipitation of $CaCO_3$ from lime or lime-soda softened water. Hot water heaters are being kept free from scale when hard high bicarbonate waters are used. Dissolved iron has been kept in solution, and the film of metaphosphate formed on the metal surfaces in the system has successfully prevented corrosion and all at concentrations of only a few ppm.

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[*A summarial statement prepared for the Reference and Data Section of *Water and Sewage Works* by Mr. Rice at our request.—*The Editors.*]

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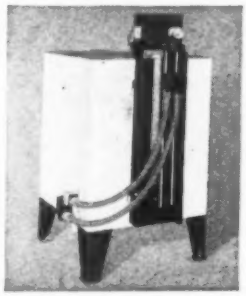

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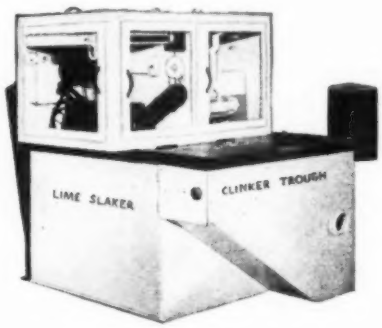

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LIME AND LIME SLAKING

By H. E. LORDLEY

Plant Manager, Dept. of Public Utilities, Richmond, Va.

LIME is one of the oldest chemicals known to civilization, yet the efficient utilization of this important chemical is impeded by lack of complete information concerning its physical and chemical properties. Since lime is used in such large quantities for softening and corrosion control in the water works field, it is amazing that so little information is to be found in water works literature on the subject of lime and lime slaking.

There Is Lime and Lime

Limestone, which is the source of the calcium oxide, will vary in physical and chemical properties in different parts of the country. Some limestone is high in magnesium which is objectionable for use in water purification plants, yet suitable for other chemical processes. Other sources are high in iron and silica, so it is well to study an analysis of the raw stone before buying lime for the water plant. At the lime plant, in the first step of processing the rock for calcining, a superoir finished product results if mud, sand, and other impurities are removed before burning. In the actual calcining step, the manufacturer can control both the physical and chemical properties of the lime, either by the type of kiln or the method of firing. Therefore, we get much lime that is either under-burned or over-burned by lack of control of the process at the kiln.

Take, for example, the old style vertical kiln using coal as a source of heat without modern control equipment, we find that the lime thus burned will slake very slowly. On the other hand, the modern controlled rotary kiln produces lime that will slake so rapidly that it actually explodes in the slaker. The author recalls an amusing incident at the Richmond Filter plant on changing from a vertical kiln to a rotary kiln lime. The operator frantically called to report that the lime slaker was out of order and making a terrible rumble in the slaking chamber. On investigation, the new rotary kiln lime was producing miniature explositions that sounded like a loose agitator blade. Also common to the old style vertical kiln is the large percentage of core as compared to practically core-free lime from the rotary kiln. So we see that all lime is not the same from every lime plant and the water works executive must choose the source of lime carefully if highest efficiency is to result.

As to Lime Properties

After the lime leaves the kiln it must be graded as to size. Quicklime can be furnished in sizes varying from a fine pulverized product, which will pass a 200 mesh screen, to a pebble lime of two inch size. In determining the proper size to use in the plant, the operator must remember that the pulverized lime offers both a dust problem and a feeding problem due to arching in the hopper. Dust can be controlled by proper ventilating system, but archhing is just a universal worry. Arching can be partly corrected by design of light weight hoppers having a bottom slope of not less than 60° from the horizontal. "Syntron" vibrators on the side of the hopper will further reduce arching, but the best answer is the use of pebble lime of size from ¼ inch to ¾ inch, provided suitable feeding and slaking equipment is available. Dust is reduced, arching in hopper is eliminated, and air-slaking of lime in storage is not as rapid with pebble lime as with the pulverized lime. Another physical property of interest is the color of the lime; white, gray, black, blue, etc. All are high in calcium oxide, and color is not a criterion of a good lime. The chemical properties and specifications are easy to determine in the laboratory. The lime should contain not less tran 90 per cent CaO, have a low magnesia, silica, and iron content, and be free of arsenic.

In many plants today, calcium hydroxide (hydrated lime) is preferred to calcium oxide, due largely to lack of information and equipment for slaking the oxide. The lime manufacturer is anxious to dispose of calcium hydroxide, since all fines from calcium oxide are converted into calcium hydroxide at the lime plant. The water works executive must remember that he pays for slaking process plus freight on water in hauling hydrated lime in preference to burned lime.

Economic Considerations

TABLE I

	Typical Calcium Oxide Lime	Typical Calcium Hydroxide Lime
Insoluble Residue	1.57	.37
R_2O_3	.48	.52
Loss on Ignition	1.34	2.19
Magnesium oxide	1.26	.50
Calcium oxide	95.33	92.00*
CaO Equivalents	96.59	70.00
Cost Price Del./Ton	$11.00	$10.00
Cost Price per ton CaO	$11.38	$14.28

*Calcium Hydroxide—$Ca(OH)_2$ or $CaO + H_2O$.

In Table No. 1, a typical analysis and price of calcium hydroxide as compared to calcium oxide shows the economic status of the two types of lime. In buying a typical pebble calcium oxide in place of calcium hydroxide, a net saving of $2.90 per ton of CaO equivalents is realized, plus higher operating efficiency. In actual plant operation the saving will be greater, since the theoretical ratio of one part CaO to 1.32 parts $Ca(OH)_2$ approaches a practical operating ratio of 1 part CaO to 1.5 parts $Ca(OH)_2$.

Another way to save money in buying quicklime is in the size of the lime. A pulverized calcium oxide (passing 200 mesh) is more costly than a standard pebble or cracked lime. The manufacturer must charge for the expensive process of pulverizing and will raise the price of calcium oxide from $11.00/ton to approximately $13.00/ton for the pulverized product. Also for large plants, the lime shipped in 15 ton cars costs more than in 25 ton cars, due to the freight rates.

With the money that can be saved by shifting from calcium hydroxide to pebble calcium oxide, an excellent feeder and slaker can be purchased and the lime treatment will become a controlled process rather than the usual "hit or miss" process. The operator should investigate and experiment with different types of lime to find the one best suited to his own plant and local conditions.

Lime Slaking Equipment and Its Operation

The operation of the slaker is not easy. Like a man learning to ride a bicycle, the problem looks simple yet requires initial research and a few hard knocks before the art is mastered.

First, and most essential, is an efficient dry-feeder. In the Richmond Filter Plant, an Omega lime feeder delivers the lime to the slaking vat at an astoundingly accurate rate. The feeder is a gravimetric batch type having a maximum capacity of 175 pound/hour and a minimum capacity of 1.75-pound/hour. The feeder has a 1,000 pound capacity, and properly designed hopper equipped with an agitator and mounted on a sensitive beam scale. The adjusting mechanism for changing the rate of feed is located in a compact unit with an alarm to indicate any trouble in feeding, such as an empty hopper or failure of material to leave the hopper. The lime feed is controlled by the vibrating plate at the bottom of the hopper. The pebble lime of size not exceeding ¾ inch size falls off the vibrating plate directly into a compact Omega Slaker located on the same floor with the feeder in a floor space of only 54 inch by 84 inch for both feeder and slaker. The unit is equipped with a water operated vapor removal system to collect dust and steam from the slaker. The water from this unit is not wasted, but is used as make-up water for the slaker or in conveying the lime to the

point of application. The slaker has a minimum retention period of 30 minutes at the maximum rate of feed and is agitated with a motor driven impeller so adjusted to give proper circulation in the chamber.

Temperature Control Important

The optimum temperature for slaking will vary both with the type of lime and the retention period in the slaker. The most efficient slaker temperature range is between 150° F. and 180° F.; above 180° F. the steam and vapor will overload the vapor removal equipment and cause corrosion of the slaker and feeder. In the Richmond plant, the optimum temperature is 160° F. using a pebble lime having a 96 per cent CaO content. In order to maintain this temperature in the shaker, with our incoming water varying between 33° F. and 85° F., an electric water heater of three No. M-240, 4,000 watt Chronolox immersion heaters was purchased for the job. The heaters are equipped with three-way switches for seasonal control and thermostats on each unit to supply makeup water at a uniform temperature regardless of the temperature of the incoming water. The heat of hydration of calcium oxide is 273 calories per gram. However, in slaking small quantities of lime some of this heat is lost and must be supplemented by heat from the make-up water.

Proper Water Ratio

In slaking lime, too little water gives a heavy slurry that is too thick to be agitated properly by the impeller, yet an excess of water increases the expense of heating make-up water and will dilute the lime so that maximum retention time in the slaker is materially reduced. Different brands of lime require variable quantities of water, but a slurry of the proper viscosity is usually obtained by a ratio range of 1 pound lime to 3 pounds water to a ratio of 1:4½. After a series of experiments at the Richmond plant, the ratio of 1:4 was adopted and gives excellent operating results.

To control the ratio, a standard water meter is connected in the supply line to the heater. The meter was geared so that the lowest dial actually reads 3 times the amount of water passing through the meter. For example, one cubic foot of water passing through the meter indicated three cubic feet on the dial. This method was devised to afford the operator a chance to check the water flow at low rates much more readily than with the regular meter dial. After checking the flow, the operator refers to a chart (see Table 2) showing the meter reading corresponding to each rate of lime feed for a ratio of 1:4, adjustment being made by the needle control valve if necessary. Example: Feed 50 lbs./hr. of lime. Requires for ratio 1:4, 50 × 4 or 200 lbs./hr. of water equals

$$\frac{200}{62.4} = 3.2 \text{ cu. ft. per hour of water.}$$

On special dial, this reads 3.2 × 3 or 9.6. On the chart opposite the rate of 50 lbs./hr. is the value 9.6 which should be the reading of the meter for one hour. The operator adjusts the valve to get this reading.

TABLE II

Water Meter Reading vs. Dial Setting for Optimum Ratio one lime to four water.

Pounds/hr. Lime	Dial Setting	Meter Reading (Cu. ft.*)
1.75	1	.3
8.75	5	1.6
17.50	10	3.4
26.25	15	5.0
35.00	20	6.7
43.75	25	8.4
52.50	30	10.0
61.25	35	11.7
70.00	40	13.5
78.75	45	15.1
87.50	50	16.7
96.25	55	18.4

*Special Meter Dial. Indicates three times actual flow.

The lime after slaking is diluted with water from the vapor removal unit and flows over a weir into the outlet line. Some lime has considerable core so that the machine is designed for core removal, without stopping the slaker operation. The core is removed to a chamber to be washed to recover any calcium hydroxide on the surface and is then dumped into a trash can under the slaker. Tests are being run on the amount of unslaked lime in the sample collected at the effluent of the slaker but are not complete enough at present to report in this paper.

Piping Lime Suspensions

No other problem seems more perplexing than that of transporting the lime to the point of application 300 feet or 400 feet from the slaker. If duplicate lines are available, one line can be cleaned with a chlorine solution while the other line is in service. However, caution must be observed to prevent the chlorine from actually attacking the iron pipe. Recently suggested is the use of "Calgon" to keep the lines free of incrustants but so far little work has been reported on this method. Many plants use rubber hose and clean the line by beating or shaking the hose. The author has found that the open trough is the simplest method of delivering the lime, provided the necessary fall between the machine and point of application can be obtained.

pH Control Simplified

So accurate is the control of pH with the new Omega slaker that in the Richmond plant we can hold a constant pH of 8.7 in our finished water. It is important to note in Table No. 3 that prior to the installation of the slaker, we had trouble in our distribution system with a decided drop in pH. Now with the present slaker we have only a slight drop and a corresponding decrease of iron in the dead-ends. We can only explain this by the fact that the controlled slaking is productive of a more highly peptized lime suspension—more nearly a colloidal suspension.

TABLE III

Station No. 34*
Location—Broad Rock Road

	Before Slaker Installation 1938	After Slaker Installation 1940
Res. Cl_2	.00	.00
Turb.	6.0	.80
Color	40	10
Alk. (M.)	23	36
Alk. (Phen.)	0	4.5
CO_2	0	0
pH	8.1	8.6
Fe_2O_3	4.0	.40

*This station is located near a dead-end.

It is hoped that water works men will study lime treatment and get more lime per dollar. Remember that often obsolete equipment can be scrapped and new equipment purchased with the saving, so why not have a modern plant at no extra cost?

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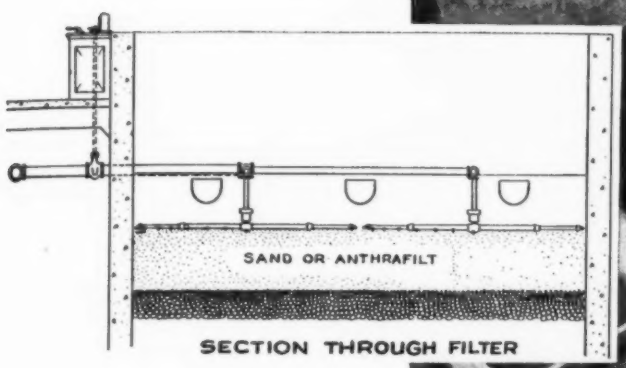

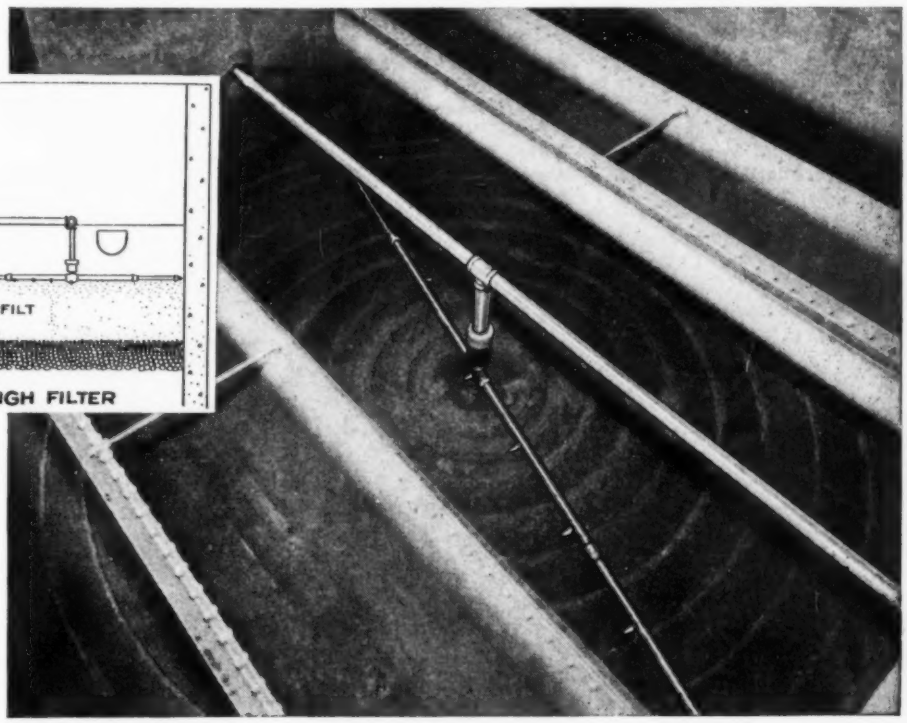

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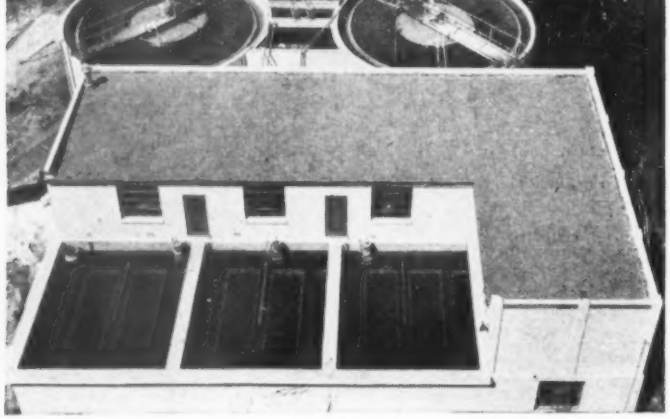

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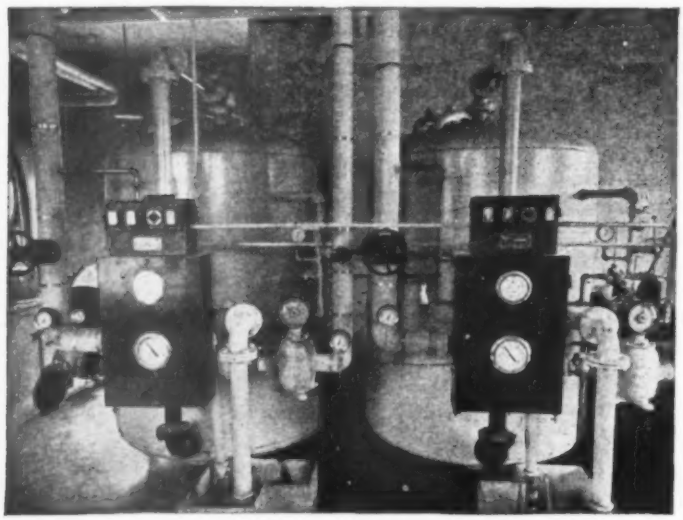

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Infilco Continuous Lime Slakers combine in one continuous, automatic process, the consecutive operations of feeding and slaking quicklime, followed by the preparation and delivery of a milk of lime mixture suitable for water purification.

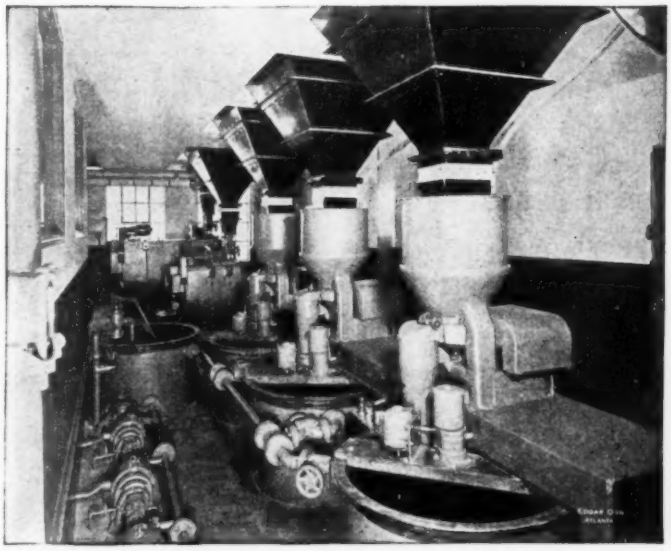
Infilco Proportioners maintain predetermined ratios between liquid flows or automatically vary the ratio of one flow with respect to another by electric, hydraulic or mechanical means.

SWIMMING POOL RECIRCULATING SYSTEMS

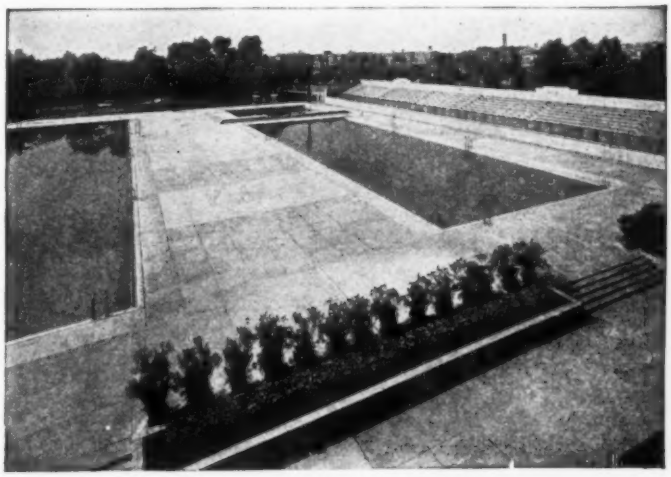
Stellar Filter Recirculating Systems have the advantages of lower installation costs, material savings in chemicals and wash water. Stellar Filters are foolproof—cannot be injured by neglectful operation.

Conventional sand filter recirculating systems, both pressure and gravity, are also popular because of distinctive Infilco design features.

Installation of 68 Operating Tables in Gravity Filtration Plant



Battery of Dry Chemical Feeders and Lime Slakers and Feeders



Group of pools protected by an Infilco Recirculating System.

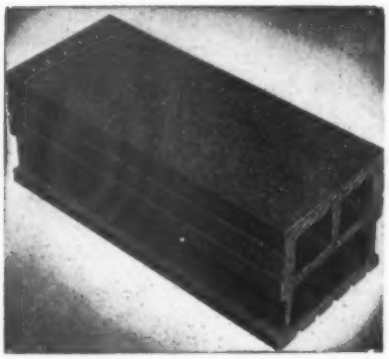
INFILCO INCORPORATED

325 W. 25th PLACE
CHICAGO 16, ILLINOIS

F. B. LEOPOLD CO., INC.

2413 W. Carson Street **Pittsburgh 4, Pa.**

Water Purification and Sewage Plant Equipment



Standard Filter Block

GLAZED FIRE CLAY TILE FILTER BOTTOM

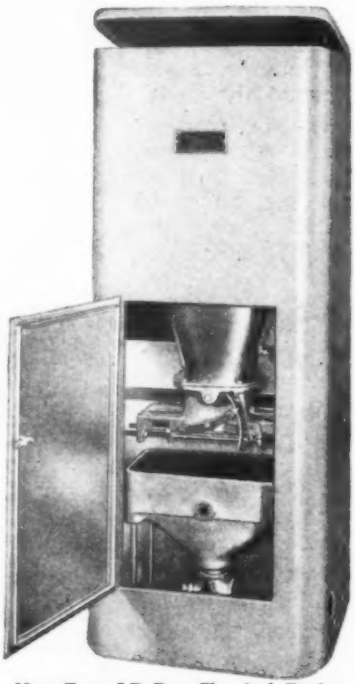
Permanent • Non-Corrosive • Economical

Designed to completely eliminate the many difficulties previously experienced with underdrain systems, the Leopold Duplex Filter Bottom has proven entirely successful in well over 100 municipal and private purification plants. Produced from de-aired fire clay—vitrified and salt-glazed, this unique filter bottom is impervious to attack by acid or alkali solutions, won't absorb any detrimental amount of water, and will last a lifetime.

The Leopold Filter Bottom is composed of separate blocks, each covering 2 square feet of filter floor and weighing approximately fifty pounds per square foot. These individual blocks are set level and firmly joined together, thus forming a strong bed that is not disturbed by wash water or air. The Leopold Duplex System assures a positive and equal collection of filtered water, reduces loss of head, and makes possible reduction of pipe and valve sizes.

Check These Features of The Leopold Filter Bottom

- Equal filtration over filter bed.
- Equal distribution of wash water.
- Blocks are combined into one heavy, strong unit.
- Vitrified, glazed fire clay blocks are practically non-absorbent.
- Glazed surfaces eliminate corrosion of blocks.
- Life of this filter bottom is extremely long.
- Highly successful performance in over 100 purification plants.



New Type LP Dry Chemical Feeder

NEW TYPE LP DRY CHEMICAL FEEDERS

The Leopold Type LP Dry Chemical Feed Machines are streamlined in design . . . streamlined in performance. Improvements incorporated in this new design include motor belt drive, adjustable vibrator and modern attractive cabinet.

The machines are of the manual adjustment type for feeding predetermined quantities of alum, lime, soda ash, nuchar or other dry chemicals at a uniform rate, once setting is made. The power unit is totally enclosed in the rear of the cabinet in a dust tight compartment, yet it is easily accessible. The operating mechanism and solution chamber are located in a separate compartment in the front of the machine.

The Type LP Feeder is compact, light in weight and durable, with all parts easily accessible. Simplicity in design keeps wear on parts at a minimum. Made in several sizes and capacities, the Type LP meets a wide variety of operating requirements.

FILTER OPERATING TABLES

Furnished in any size, combination or style desired, Leopold Operating Tables are master units for centralized control of hydraulic valves. They are designed to meet all requirements. The standard size tables are large enough for mounting any necessary gauges or samplers. All exposed metal parts can be furnished in any finish that may be desired.

In addition to hydraulic control tables, the F. B. Leopold Co. has available a complete line of equipment for filtration plants, including all types of cabinets, separate manifold headers, lubricated 4-way valves, operating plates and levers, and separate dial type valve opening indicators.

Filter Operating Table

Literature and complete details available on request.

MANGANESE REMOVAL

Experiences at Durham, N. C.

By ROBERT S. PHILLIPS*

Chief Chemist, Water Department, Durham, N.C.

THE City of Durham receives its raw water from Lake Michie on Flat River, approximately nine miles from the city. This lake will impound almost five billion gallons of water, a sufficient amount to provide the city with water even during a period of prolonged drought.

The raw water is pumped to a storage reservoir at the treatment plant through 24-in. and 20-in. mains. The storage reservoir is of sufficient capacity to hold nearly a week's supply at peak demand.

Occurrence of Manganese

The source of manganese in the Durham supply seems to be the manganese-bearing rock and clay forming the bed of the impounding lake. Long periods of impounding, due to lack of rainfall, and proper conditions for anaerobic decomposition causes relatively large amounts of manganese to be released to the raw water, the quantities being revealed in Table I.

TABLE I

Year	Percentage Normal Rainfall (Annual)	Manganese in Impounded Water (ppm) Avg.	Max.	Month of Maximum Occurrence
1935	84	0.05	0.23	August
1936	121	0.04	0.19	September
1937	115	0.03	0.07	October
1938	98	0.10	1.20	September
1939	117	0.05	0.18	November
1940	86	0.10	0.75	August
1941	54	0.27	1.40	August and October

The tabulated data reveals the occurrence of manganese may reach a maximum at any time from mid-summer to late fall. Also, unless the rainfall is not considerably above normal the manganese content may become quite serious. This is especially true when the rainfall in two consecutive years like 1940 and 1941 is far below normal. Even with normal rainfall (annual) the manganese content may reach high levels, as in 1938 when the last six months of the year were relatively dry.

Removal of Manganese by the Permanganate Oxidation Method

Since 1933 we have been removing manganese by the use of the Permanganate Oxidation Method. This method over the entire period of years has accomplished the desired results producing an effluent water low in manganese. For purposes of comparison later in this article the results of this treatment are given for August 1941 in Table II.

TABLE II

PERMANGANATE OXIDATION METHOD (AUGUST 1941)

	Parts per million Max.	Min.	Avg.
Raw Water, Manganese, (Mn)	1.20	0.60	0.82
Filtered Water, Manganese, (Mn)	0.12	0.03	0.07
Dosage, Potassium Permanganate	2.22	1.73	1.95

Although good results were indicated by the use of the lime-alum method, it would have necessitated major changes in our lime feeding and somewhat higher alum dosages would have been required. In addition it was indicated that approximately 50 per cent of the oxidized manganese would precipitate in the raw water storage reservoir, for which an adequate removal system is not provided.

Laboratory samples of raw water coagulated with ferric sulfate, $Fe_2(SO_4)_3$, had previously shown that approximately the same dosage of this chemical was required as with alum for efficient coagulation. The soluble iron in filtered samples was quite low (.05-07 ppm.). Hopkins,[1] Black[2] and Willcomb[3] report the successful removal of manganese using iron salts and lime providing the pH of coagulation is kept above 9.0.

TABLE IV

RESULTS JAR TEST
Manganese Removal with Ferric Sulfate and Lime

Raw Water Data

Turbidity	30 ppm.	Free Carbon Dioxide	8 ppm.
pH	6.7	Dissolved Oxygen	4.6 ppm.
Alkalinity	24 ppm.	Manganese (Mn)	0.7 ppm.

Jar No.	Chemical Dosages, ppm. $Fe_2(SO_4)_3$	$Ca(OH)_2$*	pH	Manganese in Filtrate, ppm.
1	25	15	7.0	0.70
2	25	20	8.2	0.70
3	25	25	8.9	0.12
4	25	30	9.2	0.04

*Note—Lime added just after floc began to form.

It will be noted that the required dosage of $KMnO_4$ was slightly more than twice the manganese content of the raw water. This ratio is typical of our experience for the past eight years when the method was used.

Early in 1941 it became evident that Potassium Permanganate crystals would not be available for use in quantities required for water treatment. This chemical, therefore, became for us the first casualty of World War II and it was necessary to seek another method of removing manganese.

Methods Put to Laboratory Test

Experimental work was immediately begun in an attempt to find a satisfactory substitute for the Permanganate Method.

As pointed out, we have a relatively large raw water storage reservoir at the treatment plant with a retention period of a week or more. An experiment was set up to determine the time required to oxidize the manganese under the influence of a relatively high pH, and to find out if the oxidized manganese could then be removed with ordinary alum flocculation.

Each of seven large bottles of lake water was dosed with hydrated lime (23 ppm.) to an initial pH of 9.3, and allowed to stand at room temperature. Each day a portion of the settled water was carefully siphoned off from one bottle. A manganese determination was made on the siphoned-off sample before treatment. Another portion of the settled sample was coagulated with 40 ppm. alum. The jar was then allowed to settle three hours, after which the supernatant was filtered (No. 12 filter paper used, discarding the first 200 mls. filtrate) and another manganese determination made on the filtrate. The results shown in Table III were obtained.

The series of jar tests using the commercial ferric sulfate "Ferrisul" and lime were set up. The dosage of the iron salt was kept constant while the lime added was varied. Each sample was allowed to mix and settle for the same periods as would be the case under plant conditions. After settling, the water was filtered and the manganese content determined.

This same test was duplicated many times and indicated that highly satisfactory removals could be expected with ferric sulfate and lime providing the pH of coagulation was kept sufficiently high. This confirmed the work of others.

The mechanism of removal of manganese by this method is apparently complicated and not very well understood. It appears to involve both chemical and physical actions. Wilcomb[3] suggests that the action takes place in two ways: first, due to the alkali

TABLE III

LABORATORY RESULTS MANGANESE OXIDATIION WITH HYDRATED LIME

Contact Time with $Ca(OH)_2$, days	0	1	2	3	4	5	6	7
pH	9.3	9.1	9.1	8.9	8.9	8.7	8.6	8.5
Turbidity, Settled Water, ppm.	60	40	35	35	35	30	30	25
Manganese, Settled Water, ppm.	0.90	0.90	0.60	0.70	0.60	0.60	0.45	0.50
Manganese, Coag. and Fltd., ppm.	0.60	0.25	0.15	0.18	0.12	0.08	0.06	0.04

*Now Supt. of Water and Sewage Treatment, Charlotte, N. C.

TABLE V
SUMMARY OF RESULTS WITH THE USE OF FERRIC SULFATE 1941-1942, DURHAM, N. C.

	1941				1942			
	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	July	Aug.
CHEMICAL DOSAGES (ppm.)								
$Fe_2(SO_4)_3$	27.1	24.2	25.0	24.0	19.7	19.4	39.8	38.2
$Ca(OH)_2$ (Primary)	27.7	26.9	24.2	22.3	6.9	0	35.9	33.8
Primary Chlorine	0.90	0.83	0.61	0.52	0.62	0.72	0.83	0.96
Ammonia	0.16	0.16	0.12	0.11	0.14	0.16	0.21	0.20
Carbon	2.0	3.2	3.7	3.9	4.8	2.5	2.8	3.9
RAW WATER (Averages)								
Temperature, °C.	21.5	20.1	14.5	9.2	6.0	5.9	20.2	24.1
Turbidity, ppm.	24	16	14	15	12	12	157	173
Hardness as $CaCO_3$, ppm.	20	20	20	21	20	21	14	15
Total Alkalinity, ppm.	25	27	24	24	24	23	15	15
Fee Carbon Dioxide, ppm.	6.3	4.7	4.5	4.4	3.1	3.4	7.6	6.9
pH	6.9	6.9	7.0	7.1	7.2	7.2	6.6	6.7
Manganese (Mn), ppm.	0.70	0.68	0.31	0.16	0.09	0.11	0.71	0.53
Iron (Fe), ppm.	2.5	1.3	1.2	1.3	1.1	0.9	7.4	9.1
pH of Coagulation	9.0	9.1	8.9	8.8	7.1	6.1	9.1	8.9
TREATED WATER (Averages)								
Temperature, °C.	22.7	20.4	14.4	9.3	6.2	5.8	23.3	24.4
Turbidity, ppm.	0.10	0.10	0.11	0.08	0.09	0.07	0.26	0.37
Hardness as $CaCO_3$, ppm.	48	47	43	41	37	37	49	49
Total Alkalinity, ppm.	37	40	33	31	28	27	27	26
Phenolpthalein Alk., ppm.	5	10	7	3	3	4	5	4
pH	8.9	9.1	9.1	8.7	8.5	8.6	9.0	8.7
Manganese (Mn), ppm.	0.03	0.04	0.02	0.01	0.05	0.07	0.03	0.02
Iron (Fe), ppm.	0.08	0.05	0.04	0.04	0.04	0.04	0.09	0.10

present the oxidation is rapid and the oxidation products are absorbed by the ferric hydrate floc, and secondly that the manganous hydroxide itself absorbs colloidal manganese particles. The largest absorption apparently is accomplished by the ferric flocs due to the fact that they predominate.

Earlier experiences at Albany and Providence had also demonstrated that ferric sulfate could be successfully and simply used as a coagulant for moderately colored soft waters containing manganese. We therefore made arrangements to employ ferric iron coagulation in Durham.

"Ferrisul" Coagulation Adopted

The commercial product used was Monsanto's "Ferrisul." This chemical is of uniform granular size feeding well in ordinary dry chemical feeders. It comes in multi-walled paper bags of 100 lbs. which can be handled readily and stored without caking. A 15-gallon crock is used as a dissolving pot, into which the ferric sulfate is fed. The crock receives a constant measured flow of water through a "Rotameter," a simple and inexpensive device for measuring low flows. The water and chemical are agitated vigorously with a mechanical agitator to insure more rapid and complete solution. The solution of ferric salt is picked up by a glass eductor and forced through an ordinary ¾-in. garden hose to the mixing chamber. The entire cost of equipping one dry feeder to feed this chemical was approximately $150.00. Very little trouble is experienced in dissolving the ferric salt and corrosion difficulties have been at a minimum.

The lime is applied to the water *after* approximately two minutes of mixing, at which point the iron floc has already started to form. After a further mix of about six minutes the water enters the settling basins.

The use of ferric sulfate was begun September 11, 1941 and continued until February 17, 1942. Again on June 28, 1942 it was employed and has continued in use to the present time. The results of these periods are summarized in Table V.

Comparing the Permanganate and Iron Methods

Comparative data on the two methods which have been used locally for manganese removal are interesting. The Permanganate Oxidation method is more costly and on the whole will not remove the manganese quite as completely as the iron treatment has. On the other hand with the ferric sulfate and lime method the hardness is higher in the finished water, due to high lime dosages required. And, at times the water must be coagulated above the Calcium Carbonate equilibrium point. Under this condition manganese removal is more complete and consistent.

TABLE VI
Comparison of Chemical Costs of Permanganate Oxidation Method and Ferric Sulfate Method

	$KMnO_4$ Method	$Fe_2(SO_4)_3$ Method
M.G. Treated	274.121	339.779
Alum, lbs.	58,200	0
Ferric Sulfate, lbs.	0	67,600
Lime, lbs.	40,200	73,950
Permanganate, lbs.	3,850	0
Chlorine, lbs.	2,508	1,908
Ammonia, lbs.	353	398
Total Cost Chemicals	$1,993.01	$1,617.19
Cost per M.G.	$7.27	$4.76

In order to give a fairly true evaluation to the relative cost of the two methods the months of August, September and October 1940 and 1941 were selected. Throughout this period both methods were used and conditions were comparable.

The Matter of pH Values

Experience throughout 1941 and 1942 in periods of manganese removal with ferric sulfate has indicated that it is not always necessary to carry the pH of coagulation above 9.0. Satisfactory manganese removals (less than .05 ppm. in filter effluent) have been obtained with the pH as low as 8.5. Rather the pH of coagulation necessary for good manganese removals will vary between 8.5 and 9.3 depending on the concentration of manganese in the raw water. The higher the manganese content in the untreated water the higher the pH of coagulation must be carried to achieve satisfactory results. This is important where industrial users require a finished water in which the calcium hardness must be kept at a minimum.

TABLE VII
Showing the Lowest pH of Coagulation to Reduce Manganese Content in Filter Effluent to .05 ppm. or Less with Various Concentrations of Manganese in Untreated Water.

pH of Coagulation	Manganese in Untreated Water ppm.
8.5	0.20
8.6	0.30
8.7	0.40
8.8	0.55
8.9	0.65
9.0	0.75
9.1	1.00
9.2	1.20

In manganese removal the Ferric Sulfate Method works very well providing the soft water treated is only moderately colored. This was true in our case during 1941. During the current season, however, the raw water contained a relatively high color—200 to 300 ppm.—in addition to high concentrations of manganese and turbidity. When the water was coagulated at a high pH for manganese removal the color of the filter effluent was 15-20 ppm. When the pH of coagulation was lowered to the acid range all of the color was removed but the manganese content remained unaffected. The choice was clear; we chose to allow the lesser of the two evils to remain in the finished water. We, therefore, delivered a water having a color four to five times its normal content.

In the meantime laboratory experiments were conducted to remove color and manganese simultaneously with the ferric sulfate method.

Various well-known color removal methods were tried in the laboratory without success. Relatively large dosages of carbon were added to the water along with the coagulants; activated carbon was suspended in the settled samples; bleaching clays with

the coagulants was tried, and sodium aluminate was tried in conjunction with the iron salt and lime. None of these methods gave positive results.

Relatively high dosages of chlorine offered the first possibility of success. The chlorine was added to the water immediately following the introduction of the coagulants, allowing the water to mix the normal time and then settled for a period of five hours. The settled water was then decanted and filtered. Color determinations were made on the filtrate. Chlorine determinations were made on the settled water.

TABLE VIII

Jar Test to Determine the Effect of High Dosages of Chlorine on Color.

Date: July 19, 1942
Raw Water Color: 250 ppm.
(Results in parts per million)
Chemical Dosages

Jar No.	Ferric Sulfate	Lime	Chlorine	Residual Chlorine 5 hours	Color Filtrate
1	40	40	0	0.00	17
2	40	40	2	0.65	15
3	40	40	4	0.75	10
4	40	40	6	0.80	5
5	40	40	8	1.10	3
6	40	40	10	2.25	0+

The above results are typical of many duplications of this experiment. Dechlorination after contact period did not affect the color.

Our plans now are to provide the necessary equipment for feeding the larger dosages of chlorine followed by dechlorination. This method for color removal in conjunction with manganese removal by ferric coagulation will be put into operation just as soon as the chlorine gas and equipment becomes available.

Summary

1. The occurrence of manganese at Durham is governed principally by the amount and distribution of rainfall throughout the entire year. The maximum concentration of manganese in the raw water may occur at any time from mid-summer to late fall.
2. Eight years of experience shows that good manganese removals can be achieved with the Permanganate Oxidation Method. Crystals of Potassium Permanganate are not available at present for water treatment.
3. Plant results with the ferric sulfate and lime method for manganese removal indicate that it is reliable and highly efficient on moderately colored soft waters. The cost of this method is considerably lower than the Permanganate Oxidation Method.
4. Where soft waters are highly colored, difficulty may be encountered in removing both color and manganese simultaneously by the ferric sulfate method. High chlorine dosages along with coagulants make possible the removal of color and manganese at the same time.

REFERENCES

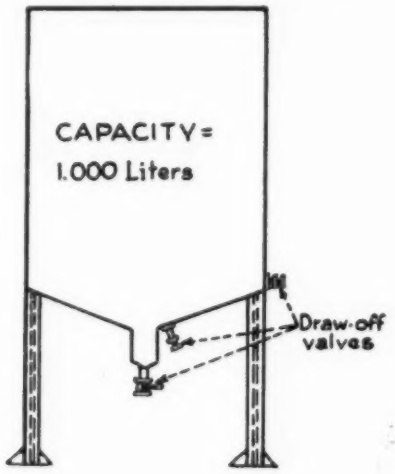
(1) Hopkins and Whitmore: Jour. Ind. and Eng. Chemistry, 22, 79, 1930.

(2) Black, A. P.: Jour. A. W. W. A., 26, 1713, 1934.

(3) Willcomb, G. E.: Jour. A. W. W. A., 31, 1703, 1939

THE DETERMINATION OF SAND IN WELL WATER*

A sample of water is collected in a two (2) liter bottle and is given a rotary motion so as to move the sand toward the center of the bottom of the bottle and to form a disk. Standards are prepared from a similar sand by placing in wide mouth bacteriological bottles, of 125 ml. capacity, measured amounts that correspond to 1, 3, 5, 10, 15, 25, 50, 100 and 200 parts per million by volume in two (2) liters of water. Water is then added to about half the height of the bottle. For standards of 300, 400, 500 and 600 p.p.m. it is preferable to use bottles of 250 ml. The standards are given a rotary motion to obtain a disk of sand in the center of the bottom of the bottle and the disks are then compared with the disk on the bottom of the bottle of the two (2) liter sample of water. It is essential to select bottles with a smooth, flat bottom of clear glass. There is no difficulty in determining whether the sand in a two (2) liter sample is more or less than one (1) part per million by volume.



TANK
FOR DETERMINATION OF SAND

During a two-year period about 4,500 daily routine determinations of sand in the water pumped from 16 deep wells of the Maracaibo water works were made and reported weekly. Following the removal of sand that accumulated in the bottom of the wells during several months of pumping, tests have been made on the water pumped to waste to determine when the sand content decreased to 5 p.p.m. before the water was allowed to enter the collecting system. A routine determination is made of the amount of sand in all samples of water from deep wells that are delivered to the laboratory of the Ministry for physical-chemical analysis.

For the more accurate determination of the amount of sand a cylindrical steel tank, of a capacity of 1,000 liters, with a conical bottom may be used. Valved outlets, as shown in the illustration, should be provided. The bottom of the tank should be elevated above the ground by legs attached to it or by an independent frame of steel angles so that a small pail or other receptacle may be placed under the lowest outlet.

The tank is filled with the water that is being pumped from the well and is allowed to settle for 30 minutes or longer, if the sand is very fine. The upper outlet is then opened to draw the water down to the cone-shaped bottom. The remaining water is then stirred with a rotary motion to move most of the sand into the small cylinder on the bottom of the tank. The water in the cone above the cylinder is then drawn off slowly through the outlet that is provided. A suitable receptacle, preferably white enameled, is then placed under the cylinder and the lowest outlet opened to discharge the water and sand. The bottom of the tank should be flushed with small amounts of water to remove any grains of sand that may remain.

After the sand has settled to the bottom of the receptacle most of the water may be siphoned off and the sand washed into an Imhoff cone of 1.0 liter capacity, graduated from 0 to 1.0 ml. in 0.1 ml. divisions. A cone with a sharp tip is preferable as more accurate readings of volumes less than 1.0 ml. may be obtained. The volume of sand should be read while it is submerged in water, in order to obtain the true volume, and then converted into equivalent parts per million by volume.

It is preferable to employ a tank with a capacity of 1,000 liters, not only to obtain a satisfactory amount of sand but also a direct reading from the Imhoff cone as 1.0 ml. of sand equals 1.0 part per million by volume. The interior of the tank should be kept well painted to prevent the addition of particles of rust to the sand and it should be thoroughly cleaned before a test is made.

*By George C. Bunker, Consultant, Panama.

THE ANTHRACITE EQUIPMENT CORP.

Anthracite Institute Building
WILKES-BARRE, PENNA.

ANTHRAFILT — an Improved Filter Medium

Trade Mark Reg. U. S. Pat. Off.

Address sales and technical inquiries to

Palmer Filter Equipment Company, 822 E. 8th St., Erie, Penna.

Anthrafilt and Its Application

Anthrafilt is made from a **selected** Pennsylvania anthracite which has been cleaned so as to have a minimum ash. This cleaned anthracite is then screened and classified hydraulically to yield sizes suitable for filter purposes.

Anthrafilt is used as a mechanical aggregate in place of sand and gravel in all kinds of filters, i.e.

Rapid gravity filters	Slow filters
Swimming pool filters	Pressure filters
Hot process softening filters	Oil removal filters
Cold process softening filters	Alkali filters
Mechanical sewage filters	Acid filters
Revolving screen filters	Sewage sludge-beds
Portable filters	Household filters

The entire filter bed may be made up of Anthrafilt of various sizes or only the top layer may be of Anthrafilt over the usual bed of gravel.

Advantages of Anthrafilt

High Filtration Rate: Rates up to 200 mgad are obtainable with Anthrafilt without sacrifice of effluent quality. The accepted average rate with sand is 125 mgad.

Long Filter Runs: The larger, irregular Anthrafilt grains allow less clogging with algae or other fibrous substances. The angular grain surfaces provide greater entrapping areas for the removal of turbidity and bacteria. The differences in shape and specific gravity of the grains, by decreasing classification after back washing, allow the whole depth of the filter bed to be active in the filter operation.

Low Wash Water Costs: The long filter runs and consequent less frequent washing and the low degree of packing after back washing combine to cut the usual wash water requirements at least in half.

No Cementing with Lime, Manganese or Iron: Lime will coat on Anthrafilt but this coating readily flakes off without any cementing action if filters are properly operated. Manganese and iron salts precipitated on Anthrafilt by the lime treatment are easily removed by back washing.

High Degree of Expansion: 50% expansion is obtained with half the back wash velocity required for sand.

Coarse Sizes Can Be Used: The irregular shape and natural filtration characteristics permit the use of much larger sizes with equal results.

Gravel Beds Stay Flat: No ridging or mounding and less cracking and pulling away from side walls during operation.

Low First Cost: Lighter weight per unit volume balances the cost differential between Anthracite and sand and saves on freight and handling charges.

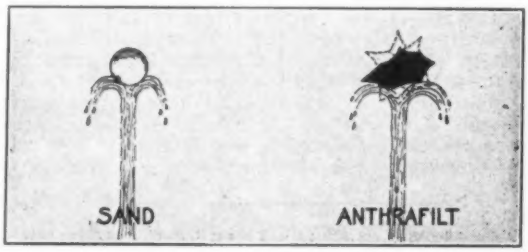
Filtration Characteristics

Anthracite granules are irregular shaped and of varying densities. Weights run about 53 pounds per cubic foot as against 100 pounds for sand. The irregular shapes provide greater porosity resulting in more rapid filtration with no loss in character of effluent and with considerably less loss of head.

The variable densities of Anthrafilt particles cause less classification after back washing and the lighter, irregular shapes allow easier expansion and more effective cleaning.

The greater bulk of Anthrafilt requires less tonnage per filter bed. Attrition losses amount to less than 0.25% per year.

Anthrafilt successfully removes lime, iron and manganese salts and will not form a permanent coating under proper conditions of treatment and filter operation. Lime flakes off easily without cementing and the iron and manganese salts are readily removed by back washing. Oil is easily removed from boiler water after coagulation with ferric sulphate, ferrous sulphate or alum. Anthrafilt can also be used for the filtration of acetic and sulphuric acids and caustic soda solutions. Being insoluble in alkaline waters, Anthrafilt produces silica-free water in hot process water softening.



Upward impulses of water produces greater agitation of angular grains, hence Anthrafilt beds are cleaned more effectively.

Anthrafilt No. 1 is usually made in sizes ranging from effectixe size of .60 m.m. to .80 m.m. the uniformity coefficient is less than 1.75 in all sises. We can make smaller sizes or larger sizes, but for special cases only. The remaining sizes of Anthrafilt are as follows:

No. 2. Anthrafilt—3/32"x3/16"
No. 3. Anthrafilt—3/16"x5/16"
No. 4. Anthrafilt—5/16"x9/16"
No. 5. Anthrafilt—9/16"x13/16"
No. 6. Anthrafilt—13/16"x1-5/8"
No. 7. Anthrafilt—1-5/8"x2-7/16"
The above sizes have 20% tolerance.

ANTHRAFILT ECONOMY

When Used in Existing Filtration Plants

Increased capacity with same plant
Decreased volume of wash water
Lower maintenance cost through elimination of ridging, mounding and air binding.

When Used in New Filtration Plants

Lower plant investment
Lower cost per cubic yard
Lower wash water costs
Lower maintenance costs

CONTROL OF MICRO-ORGANISMS AND AQUATIC VEGETATION

BY L. F. WARRICK
H. E. WIRTH AND W. VAN HORN*
Madison, Wisconsin

PUBLIC interest in clean waters has manifested itself in many ways in recent years. Pollution control agencies have been established in many states, and every session of Congress for the past decade has been called upon to consider legislation relating to this subject. Stream clean-up programs have been inaugurated. In rapidly increasing numbers municipalities have installed modern sewage treatment works. Industries have either proceeded or are being induced to develop and install methods for utilization or treatment and inoffensive disposal of pollutional wastes. The objective of all these efforts is to restore polluted water courses to proper sanitary conditions and to keep them as clean and attractive as possible.

The purpose in discussing the control of aquatic nuisances by chemical methods is to present another phase of the general problem of keeping waters clean and attractive, aside from treatment of sewage and trade wastes. The intent is to set forth, briefly, difficulties created by superabundant growths of algae, rooted weeds and certain obnoxious growths in lakes and streams, and to follow with a review of developments in overcoming some of these difficulties.

Algae may be popularly described as microscopic, one-celled plants produced in immense quantities in shallow water areas under favorable seasonal conditions. Growing in reservoirs, they achieve notoriety as the cause of nauseating tastes and odors in drinking water. The effectiveness of copper sulphate in suppression of algae growths has long been known, but there are certain important considerations involved which must be met in large-scale applications of such treatment to assure satisfactory results. It is also proposed to describe some of the studies that have been carried out under the general supervision of the Wisconsin Committee on Water Pollution through its Committee on Chemical Treatment of Lakes and Streams, with attention being called to methods for more definitely controlling dosages of chemical to prevent unnecessary damage to fish and other aquatic life, particularly fish foods.

General Problems

Under certain conditions, not well understood at present, algae multiply so rapidly that in a number of lakes and streams their presence gives the water a pea-soup color and consistency. These algae "blooms" naturally detract from the enjoyment of the waters by cottagers, bathers, boaters and fishermen because of the stench produced during the decomposition of such masses of algae. Similarly, many weed beds, heretofore ignored or regarded as desirable for the maintenance of good fish population, have become a nuisance and hindrance to the more popular recreational uses of lakes and the development of residential areas or summer resorts around bodies of water.

Rooted plants growing in abundance in recreational areas have caused much trouble for resort and other property owners. No one enjoys using a beach populated by a dense growth of weeds. Boating is difficult when weeds become wrapped around oars or propeller. Offensive odors result when the plants decompose after they have been cut and washed ashore, or have died in the fall.

One-Man-Power Barge
Applying copper-sulphate treatment to a shallow waterway to control odors from algae growth.

One of the Smaller Outfits for Copper Sulphate Spraying of Small Lakes

Some people are allergic to aquatic plants and develop rashes when they swim in certain weedy areas. Another problem, closely allied to allergy, is caused by a microorganism (*cercariae*), the effects of which are commonly referred to as "swimmer's itch." Outbreaks occurring in some bathing areas have annoyed and alarmed persons both directly and indirectly affected.

There is also an economic aspect that must be considered. The decreasing value of real estate bordering infested lakes is a direct loss to owners and eventually may have a serious effect on tax rolls. Industrial processes have been interfered with at times by algae growths, which have hampered production or caused loss through spoiling the product. For instance, paper mills have had objectionable odors imparted to paper and boxboard used for packaging foods and confections by large amounts of algae growing in the water supply used by the mills. There are instances on record where the death of cattle and other livestock has been attributed to the presence of algae in the drinking water. All of these conditions have resulted in efforts by property owners and others concerned to control these algae, weed and other growths considered aquatic nuisances.

Control Measures

Efforts to eliminate unwanted vegetation have generally utilized manual or mechanical methods. Under-water cutting machines have been made available for cutting rooted weeds. The manually-operated equipment consists of a ribbon steel saw which can be drawn back and forth across weedy areas in the manner of a lumberjack's saw. Power cutters mounted on barges, patterned after harvesting machinery, have been developed. Tractor or truck drawn cables have been used to remove littoral vegetation. (See WATER WORKS AND SEWERAGE for March, 1942—"The Control of Water Chestnut Growths.") Cut material must be collected and removed for burning, burial or disposal on land at isolated places. Otherwise, it will drift to shore, decompose and cause an odor nuisance. Dredging generally effects more lasting results than weed cutting because of removal of the roots.

At best, all of these are laborious, time-consuming methods. They are effective in solving only a part of the general problem of aquatic nuisance control. They do not take care of troubles caused by excessive growths of algae or small free-floating plants.

Early Studies:

Investigations of chemical treatment were started at an early date to devise practical procedure for taking care of all phases

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of the problem. Tastes and odors in water supplies having been identified with growths of algae more than sixty years ago, attempts were first directed to overcoming this difficulty. Studies were undertaken by the Bureau of Plant Industry, U. S. Department of Agriculture, in cooperation with the Massachusetts State Board of Health, and in 1901 Moore[1] successfully worked out a procedure for destroying algae with small quantities of copper sulphate. Further studies of the application of this method were reported in 1904 by Caird[2] at Elmira, N. Y., Carroll[3] at Butte,

"Stern Wheeler"

This barge, for the control of both algae and rooted weeds, is used by a Sanitary District set up by a Wisconsin town for the primary purpose of applying preventive treatment in the control of aquatic nuisances.

Mont., and Fletcher[4] at Hanover, N. H. Results of the treatment of the reservoir of the Winchester, Ky., water supply were reported in 1905 by Moore and Kellerman[5]. Additional information was presented by Moore and others[6] during the same year in a symposium conducted by the New England Water Works Association. Prince[7], Caird[8], Kellerman[9], Huff and House[10], Hale[11] and others have added much to the knowledge on this subject following this early work. Ample experience has been gained to date to definitely establish the role of copper sulphate treatment in the control of algal tastes and odors in water works practice.

Chemical suppression of unsightly and ill-smelling algae scums, as a nuisance control measure in lakes and streams used for recreational purposes, has largely developed during the last two decades. A pioneering study in this field was conducted by Alvord[12] at Madison, Wis., who recommended in 1920 that copper sulphate be applied, under competent supervision, in sufficient amounts to maintain the lake waters in question in a clean and attractive condition. Experiments were continued for the next few years by the city and the University of Wisconsin, but it was not until 1925 that suitable procedures and equipment had been devised to carry out an effective treatment program.

Domagalla[13] has described the spraying facilities found most satisfactory, consisting essentially of a solution tank for copper sulphate and a power sprayer mounted on a scow. Use of this equipment supplanted the original practice of dragging burlap bags of the chemical behind a boat, with both more efficient and economical results. Domagalla also mentions his use of arsenical compounds for destruction of a variety of water weeds. In continuing work on the lakes at Madison and elsewhere up to the present he has further improved the technique of treatment and done much toward placing the control of aquatic nuisances on a sound basis.

Regulatory Measures

It has been inevitable that controversy would arise regarding the placing of chemicals like copper sulphate in waters frequented by fishermen. There have occurred certain instances where misguided enthusiasts, working on the theory "if a little does good, more should do better," have destroyed fish. These mistakes have resulted in demands that all practices of chemical treatment of this nature be prohibited or there be inaugurated adequate regulatory measures.

In Wisconsin, as a result of executive order, a committee was established in 1938 to review the whole problem of algae and weed control and recommend suitable policies and procedures of administration. It was finally determined that the work should be carried out on a continuing basis as one of the functions of the State Committee on Water Pollution, a statutory agency vested with broad powers for controlling pollution.

The 1941 legislature passed an act calling upon the Committee on Water Pollution: "To supervise chemical treatment of waters for the suppression of algae, aquatic weeds, swimmers' itch and other nuisance-producing plants and organisms." To this end the committee may conduct experiments for the purpose of ascertaining the best methods of such control. It may purchase equipment and may make a charge for the use of the same and for materials furnished, together with a per diem charge for any services performed in such work. The charge shall be sufficient to reimburse the committee for the use of the equipment, the actual cost of the materials furnished, and the actual cost of the services rendered plus 10 per cent for overhead and development work. The legislature also passed an amendment to the Town Sanitary District Act authorizing the control of aquatic nuisances by a Sanitary District and providing means for financing such measures.

The Wisconsin Committee on Chemical Treatment of Lakes and Streams consists of an attorney of the Wisconsin Public Service Commission; the chief biologist of the State Conservation Department; the chief chemist of the State Laboratory of Hygiene; and the State Sanitary Engineer of the State Board of Health. Its principal activities have been to obtain all facts available on chemical treatment, institute research to determine procedures to prevent unnecessary damage to fish and fish foods, and establish supervisory control over treatments so that chemicals would be scientifically applied rather than promiscuously used in the waters of the state. Laboratory and field investigations have been carried out with the aid of both special personnel and technical staffs of departments concerned.

The committee studied the effect of chemicals employed on various species and sizes of fish and some of their food material to determine toxic concentrations. Some of the results obtained are mentioned later. Spraying devices were tested and improved, dusting and broadcasting equipment for spreading chemicals was tried, and various other facilities to aid in obtaining reasonably good control over dosages have been studied. The biological laboratories at state fish hatcheries, and facilities at the University of Wisconsin were made available for this work. The committee has also developed a permit system in the supervision of treatment activities.

Apparatus and Treatment Procedure

During the summer of 1939 the Committee on Chemical Treatment of Lakes and Streams authorized treatment and special studies of several lakes in the state using copper sulphate in order to suppress objectionable algae growths. The first treatments were made with improvised apparatus which was largely of an experimental nature and was later changed somewhat to provide a more convenient means of treatment. A 50-gallon wooden barrel having a partition extending from the top to within six inches of the bottom was used as a solution box. Copper sulphate crystals of approximately pea size were placed in the barrel and hot water from the cooling system of the boat engine discharged into the barrel on one side of the partition. This liquid then passed downward under the partition and into the copper sulphate crystals and upon reaching the other side of the partition was practically saturated with copper sulphate.

A motor-driven portable pump was obtained and placed in the boat for the purpose of spraying the copper sulphate solution. Water was drawn from the lake and a small amount of the concentrated copper sulphate solution was introduced into the suction side of the pump. In passage through the pump the solution mixed with a comparatively large amount of lake water before being discharged through two 1-inch rubber hoses equipped with nozzles which were manually operated to spray the dilute copper sulphate solution onto the surface of the lake.

The first treatments were controlled by varying the speed of the boat and the amount of copper sulphate crystals placed in the solution barrel. It was found that this method would dissolve approximately 200 pounds of copper sulphate per hour. Later developments and refinements in the treatment process included the construction of a speedometer which gave an accurate measurement of the speed of the boat and also the installation of a small Venturi meter which measured the amount of copper sulphate solution being discharged into the suction side of the pump. Later another baffle was placed in the solution barrel and it was found that it was possible to obtain a saturated solution of copper sulphate which did not vary appreciably in strength.

Tables were also made, indicating the amount of copper sulphate necessary for the treatment of a given body of water and

it was found that the most satisfactory method of controlling variables was by changing the volume of saturated copper sulphate solution fed into the system. In this way the concentration of copper sulphate sprayed onto the surface of the water could be varied in proportion to the speed of the boat so that the resulting concentration in the lake water could be maintained at any desired figure. In hard-water lakes the usual amount of copper sulphate added was equivalent to 1 p.p.m. in the top four feet of water. At the present time the apparatus is being further developed by the construction of a portable photolometer with which the concentration of copper sulphate solution can be accurately checked.

It has been found that one of the most essential steps in treating a lake to suppress algae growths is a survey to determine the concentrations of algae, the areas of the lake which can be most beneficially treated, the hardness and other characteristics of the water, the depth of water, and type of algae present. With this information it is possible to accurately estimate the amount of copper sulphate needed and the areas which should be treated.

At the present time the Committee on Chemical Treatment of Lakes and Streams has apparatus available for the treatment of such lakes which can be used on a rental basis by any interested party under direct supervision of representatives of the Committee. The Committee furnishes supervision of such treatment in order that satisfactory control over the amounts of copper sulphate can be maintained. A charge is made to lake improvement associations and other groups sponsoring the work. The program has been quite successful in reducing nuisances from prolific algae growths and has not appeared to be harmful to fish life.

Toxicity of Copper Sulphate to Fish and Fish Food

Numerous tests have been made to determine the copper sulphate dosage required to kill fish. The relation between alkalinity and toxicity of this chemical to fish was shown by experiment under controlled conditions in aquaria, measured in time required to cause death. The relative toxicity of 25 p.p.m. copper sulphate to brown trout fry varied inversely with the alkalinity. With 6 p.p.m. of alkalinity in the water, all fish died in 2.5 hours; with 248 p.p.m. alkalinity there were four survivors after 12.5 hours.

The large discrepancy between the accepted toxic dose for bass (2.1 p.p.m.) and that found in recent studies (160 p.p.m. or more), was explained as follows: The toxic dose of copper sulphate for several species of fish (black bass, yellow perch, sunfish, and bullheads) was determined in distilled water. The values so obtained agree well with those appearing in the literature. Thus, the conclusion must be drawn that the tests of previous investigators had been made in either distilled water or in very soft waters.

Chemical examination of hard water treated with copper sulphate disclosed the fact that the calcium carbonates and bicarbonates react with the copper sulphate producing insoluble basic copper carbonate, which in hard water was shown to be non-injurious to fish. The rate of removal of the copper from solution as a relatively harmless precipitate is not, however, an instantaneous reaction. Several attempts were made without very reliable results to fix exactly the actual rate of removal. In general, however, it was found that the majority of the copper is removed from solution in about one-half hour.

The fish used in the above tests were less than one year old and averaged about 4 inches in length. In order to ascertain whether very young fish were extremely sensitive to the chemical, a test was made with bass fry one and one-half months old and averaging one inch in length. Again the mineral matter of the hard water offered a protective action and these fish were not affected until a concentration of 80 p.p.m. was reached. The results indicate that the young fish are more susceptible than older ones, but that there is still a large margin of safety present between dosages which destroy micro-organisms and those which will kill young fish.

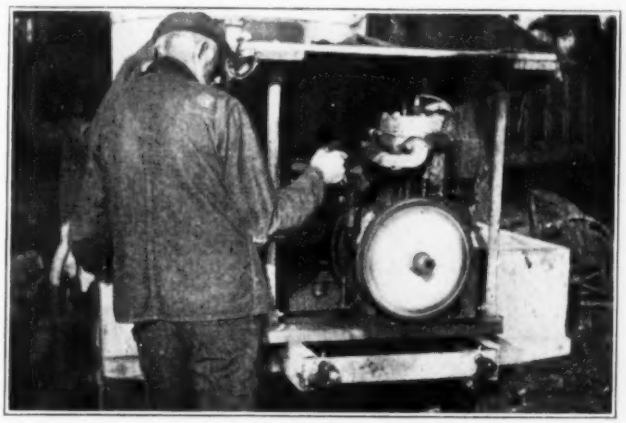
Results of the fish studies with various concentrations of copper sulphate have indicated: (1) The toxic doses of copper sulphate recorded in the literature for the various species of fish are correct only for distilled or very soft waters. (2) The mineral salts of hard waters remove copper from solution as an insoluble precipitate, thus increasing the amount of chemical which may be added before fish are killed. (3) The fatal dose for young large mouth black bass in hard water is 80 p.p.m.; for larger bass approximately 160 p.p.m. (4) Very short exposures of fish to high concentrations of copper sulphate produce high mortality.

Survival of Fish Food Animals in the Presence of Copper Sulphate:

A most important question concerns the effect of copper sulphate on animals other than fish; are certain kinds eliminated from a lake or so diminished as to reduce the fish food supply? A number of experiments undertaken on certain type organisms in various concentrations of copper sulphate showed the following: Rotifers survive low concentrations but were killed in concentrations ranging from 15 to 1,000 p.p.m. within a 30-hour period. Cyclops survived 4 to 15 p.p.m. concentrations for 88 hours, the length of the test. The common flat worm was found sensitive to a fairly low concentration of copper sulphate. All were dead within 24 hours in concentrations of 10 to 50 p.p.m. Amphipods proved to be very sensitive, even to low concentrations, and undoubtedly are reduced in numbers when lakes are given copper sulphate treatment. Species that proved to be very resistant to copper sulphate included the isopods, damsel fly nymphs, water beetle larvae, and caddis fly larvae. In general, experiments and field observations have indicated that most animal organisms are far more resistant to low concentrations than are the several species of nuisance algae. Therefore, the animal organism population will not be seriously reduced or permanently damaged by the methods now in use for controlling algae nuisances.

Relation Between Algal Density and Copper Sulphate Dosage

An experiment was set up to obtain some idea of the rate at which typical algal cells absorb copper from solution. Living plants of *Microcystis* were placed in water of 154 p.p.m. alkalinity in a 5-gallon aquarium. Copper sulphate was added to make a 2 p.p.m. solution. Samples for copper determination were then taken at intervals. The conclusion was that "the amount of copper removed from solution by *Microcystis* is extremely small." Hence, for the control of this organism it appears it should not be necessary to increase the concentration of copper sulphate above 1 p.p.m. when treating lakes which have passed into an advanced bloom stage.



A Trailer Unit
Motor powered spraying outfit for lake treatment mounted on a trailer.

Swimmer's Itch

In the matter of "swimmer's itch," tests have shown the possibility of control of this difficulty by chemical treatment under certain circumstances, and accordingly the committee has been giving consideration to control of this nuisance as one important phase of its activities.

The most serious aspect of "swimmer's itch" probably is its detrimental effect on resort business. So far as has been determined, all cases of this type of skin irritation that have occurred in Wisconsin can be attributed to a single cause; namely, the accidental penetration into man's skin of the larvae of schistosome flukes (flatworms). These larvae, having developed in snails, emerge into the water where they will die unless they encounter the proper bird or mammal in which to complete their development. The infested birds and mammals in turn scatter eggs of the worm, which hatch and undergo development when they enter the snails.

This knowledge of the life cycle of the organisms causing "swimmer's itch" (more exactly termed schistosome dermatitis) is important because it suggests possible points of attack in

attempts to control the disease. Since the presence of this disease is of some economic importance to the state, studies have been carried out with the object of determining possible methods of controlling or combating the condition. The more important facts that have been learned to date will be presented here. A more extensive account of the researches substantiating the statements made have been published by Brackett (see References 14-19 inclusive).

Investigations in Wisconsin during the summer of 1938 and reliable accounts of the occurrence of this disease in earlier years indicate that, with the exception of a few localities, only a relatively small number of individuals are ordinarily affected. With such being the case, it is obvious that control measures that are at all extensive or expensive cannot usually be justified. In these places, where other control methods are impracticable, it may prove helpful to bathers if they rub themselves briskly and thoroughly with a towel immediately upon coming from possibly infected water and before the water has time to be removed from the skin by evaporation. Apparently most of the larvae penetrate while evaporation is taking place in the thin film of water covering the body. The rubbing crushes them while they are still on the surface of the skin before they have penetrated.

Particular attention is called to this prophylactic measure because it is simple, costs nothing, and puts the responsibility directly on the persons concerned. Likewise it is a procedure that can always be used anywhere, regardless of varying environmental conditions, a fact which is not true of other control measures to be discussed. At present the possibility of controlling the parasite during the mammal phase of its cycle is remote. Methods designed only to kill the larvae in the water would be temporary since the snails may shed great numbers of the larvae for a period of several weeks.

Chemical Solution Tanks and Power Spraying Unit in a Barge

Dosages are controlled by regulating strength of chemical solutions and varying speed of barge travel indicated by a pendulum speedometer. Dosages are checked at intervals by colorimetric tests.

To eliminate the host snails, involved in outbreaks of "swimmer's itch," is undoubtedly the most effective and permanent method of preventing this annoying condition. It is evident from the absolute essentiality of snails in the life cycle of this parasite that the snail stage is a vital point of attack. The bulk of the studies have therefore been made on snails and their relationship to the disease. Amounts of copper carbonate as small as 3 lbs. per thousand square feet of bottom area have been used to kill the snails.

Rooted Weed Control

The elimination of rooted weeds involves the use of sodium arsenite which is a more dangerous chemical than copper sulplate. Arsenical compounds are considered as poisons. A dilute solution is sprayed onto the surface of the water over the plants. The plant is killed by the arsenic absorbed through the leaves and carried to the roots. The dosage depends on the weeds to be destroyed. The treatment is permanent until the area is reseeded. Experiments showed that fish (bluegills) can live at least six days in a solution that contains sufficient arsenic to kill plants. Further research showed that large mouth bass survived 232 hours in the presence of 6 p.p.m. Survival of 87 per cent was had in a 10 p.p.m. solution. Croppies and bluegills were unable to stand a 15 p.p.m. solution of arsenic.

During the summers of 1940 and 1941 the Committee on the Chemical Treatment of Lakes and Streams conducted a series of experiments in order to determine the practicability of using "Benochlor" products for the eradication of rooted aquatic weeds in the state of Wisconsin. Three Benochlor preparations used were Benochlor 3, Benochlor 3 T.D., and Benochlor 3 Special.

The following results were submitted as a progress report. Twenty-four test areas were set up in six bodies of water, some area being enclosed by a canvas fence supported by net floats and weights at the bottom to hold the fence in place. This permitted a definite control of the area treated.

In no cases were very promising results obtained with Benochlor 3.

In concentrations as low as 100 p.p.m. Benochlor 3 T.D. was effective in killing *Vallisneria, Potamogeton, Anacharis, Chara,* and *Myriophyllum* within 20 days. Further investigations will be made to determine the most desirable concentration to use.

In concentrations as low as 40 p.p.m., Benochlor 3 Special was effective in killing *Anacharis, Potamogeton, Zannichellis, Myriophyllum* and *Vallisneria* in 15 days. For Benochlor 3 Special or Benochlor 3 T.D. the weeds mentioned as being killed were the only ones present in the treated areas. These investigations did not determine the toxicity of these compounds to aquatic fauna.

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A COPPER SULPHATE DISTRIBUTING CRAFT*

By SIDNEY S. ANTHONY

Supt. & Engr. Augusta Water District, Augusta, Me.

AT MORE or less frequent intervals the supply for the Augusta Water District, Carleton Pond, has become infested with numerous types of algal growths which have been very bothersome. Various methods of reservoir treatment for algae control have been tried, but with unsatisfactory results.

The method of dragging bags seemed to be rather inefficient, since narrow streaks of water were heavily dosed, and there was no way to be sure that the entire surface was being treated.

A Work-Boat Equipped for Trial

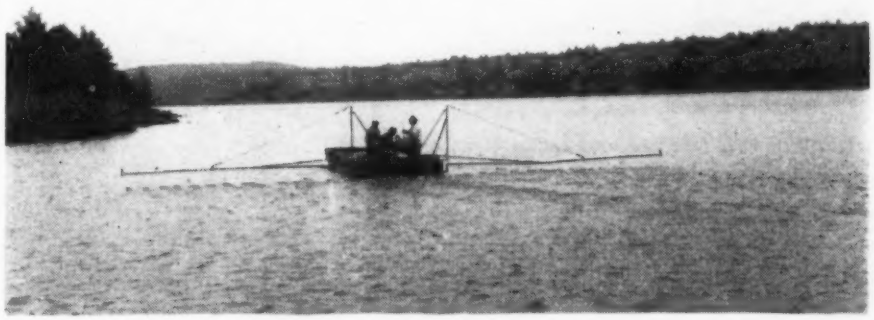
We had a work-boat, square ended and about 15 feet long and four feet wide, on which could be mounted a 9 hp. outboard motor and which would travel nearly ten miles per hour. I had a distributor pipe made up about 12 feet long, consisting of short pieces of 2 in. pipe and 2 in. x ½ in. tees with ½ in. nipples in the side outlets. A hose connected this distributor to a 3 in. gasoline ditch pump, with a length of suction hose thrown over the side. A smaller hose ran from the suction tee to the bottom of a half section of a galvanized range boiler. A small hose also led from the discharge of the pump to the other end of the range boiler. The procedure was to start the outboard motor and the ditch pump, which delivered part of its water to the boiler and part to the distributor. A man standing beside the boiler scooped copper sulfate crystals into the boiler and stirred the mixture which then passed through the pump and to the distributor pipe. The scheme worked pretty well except for two things: the boat was so small and low in the water that the slightest chop scared the operators to death; and also the crystals dissolved too slowly. We tried again, this time with pulverized copper sulphate. The solution rate was satisfactory, but even a slight breeze filled the air with dust and irritated the eyes, nose, and throats of the operators.

Distribution by Air Blowing Tried

Then we tried a small blower made up with a gasoline engine drive to spread the powder, but the powder caked in the hopper of the blower and would blow back, or rush out in chunks, or not at all. The blower was intended to cover a much larger area of pond surface, but wind conditions had to be about perfect, which they seldom are when you want them to be, so we gave up the idea of broadcasting from the blower and went back to the spray method.

We Design a Craft

We knew that a craft large enough to be safe, and large enough to carry a good supply of copper sulfate and the equipment necessary was essential, so such a craft was designed. I knew what was wanted, but being no yacht designer the sketch I took to the boat yard drew a few laughs and suggestions of changes. The changes were made and the craft, which we christened "The Bluestone," was built, as you see in the accompanying pictures. It is 24 feet long, 8 feet wide and 2 feet deep, with an undercut bow which I hoped would cause it to move more easily through the water. It is built of fir planking, oak ribs and hard pine rails and weighs about two tons without equipment. It will support about 4 tons of load with an 18 inch draft. Its cost was $475.



The Coppering Craft—"Bluestone"—Showing 50 foot Distributor Spread.

Piping Arrangements and Pump

The piping arrangement seen in the pictures was first placed across the bow, but the extra weight made it ride low at the bow and high behind, so it was shifted to the position across the stern. The original piping was galvanized iron which lasted about one season. When used early this year corrosion had damaged the threads to the point where the piping fell apart, so the entire piping has been replaced with red brass pipe and fittings.

Each hose line is valved so that the flow of water can be diverted to flush various parts of the system, stop flow to some parts, or to pump out the scow when it is filled with rain water.

The pump used is a standard make 3 in. self priming ditch pump reduced to 2 in. inlet and outlet; suction and discharge hose is 2 in. wire-wound suction type hose. The water supply to the mixing tank is 1¼ in. high pressure hose. A short piece of this same hose is used as a suction line for pumping out the bilges when required.

Performance

A crew of three men can apply more than 4,000 pounds of pulverized copper sulphate per eight hour day, one man operating the outboard motor, one scooping powder into the tank and the other stirring the mixture. They take turnabout at these jobs so that there is no monotony.

The helmsman keeps track of the course and sights on a tree or bush on the opposite shore about 100 feet from the last cross trip in that direction, and then turns for the other shore. After crossing back and forth the course is changed to longitudinal trips. First, however, when starting treatment, two or three trips are made around the shores to reach the shallows and provide turning areas without missing any part. The solution applied contains about 1% copper sulphate.

The pond is divided into three parts according to volume of water in each part. We have depth soundings which indicate a total volume of water of about 1,800,000,000 gallons of which 725,000,000 gallons are available above the intake. Treatment is based only on the available water, on the theory that the sinking copper sulfate solution does its work as it sinks.

Buoys are placed at these three division points and sufficient copper sulfate is loaded in the scow to treat one of the thirds, after which a return is made to the storehouse for another load.

Arrangement and Operation of Floating Equipment for Dissolving and Feeding Copper Sulphate.

Effectiveness

The effectiveness of this method of copper application was impressively demonstrated in the late Summer of 1945. A rapid increase in the number of standard units of organisms during late July and early August indicated treatment would be required and it was on V-J day that treatment was started. It was completed the next day.

From September of 1945 until April, 1947, weekly examinations of the water taken from the gate well showed not more than ten of any variety of organism present, and many of the reports showed a long string of zeros, with notations that "occasional" this or that was present in the water.

COPPER SULPHATE

Some Aspects of Its Usefulness and Methods of Application

By PAUL WEIR
Genl. Mgr., Water Dept., Atlanta, Ga.

THE utilitarian value of copper sulphate in the water purification plant has now come of age and it should be considered in the same light as other chemicals in the water treatment field. It should be applied by calibrated equipment of a permanent nature and not promiscuously fed through a dilapidated contraption. Proper equipment means effective and economical control.

Copper sulphate is essentially chemically pure, 25% of its weight being pure copper. It can be secured in a variety of crystal sizes from large lumps to fine powder. It is conveniently packaged in wooden barrels and 100-pound bags.

Copper is usually applied at two principal points in the purification process:

I. Open storage reservoirs.
II. In conjunction with other purification chemicals.

Methods of Application

Alone

The following four methods are most generally used in treating open storage reservoirs, depending upon their size, intensity of organism, propagation, and available equipment.

(a) Porous bag suspension.
(b) Perforated dispersion box or basket.
(c) Solution spray—(surface atomization).
(d) Pulverized crystal blower—(surface dusting).

One of the simplest methods of copper sulphate application consists in the placing of about 25 pounds of the crystals in burlap bags, or porous baskets, and attaching them to a boat, either astern or at the sides, so that they will drag in the water to be treated. Additional refills may be used as needed. It has been frequently suggested that the boat should follow a definite course, one series of parallel paths from 20 to 50 feet apart, and if the reservoir is deep, a second course should be made at right angles to the first in grid-iron formation.

Another method has been developed at Atlanta, whereby the shore line and areas too shallow for boat-travel may be treated in an effective and economical manner. An inflated automobile inner tube is laced with rope or heavy twine, in a spider web or net effect. This floating basket supports a 25-pound bag of copper sulphate. A 25 to 50 foot sash cord, attached to the inner tube, permits a person to pull this device easily from the shore line. Atlanta has two raw water impounding reservoirs with a total surface area of 60 acres. Each reservoir has approximately 6,000 feet of shore line which requires 150 pounds of copper sulphate thus applied each month.

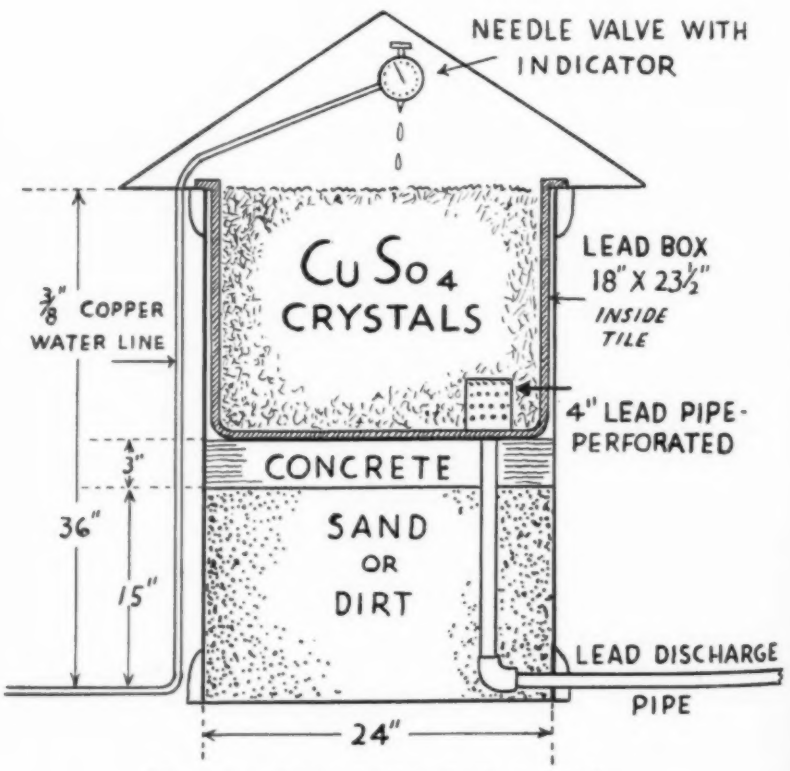
There are certain circumstances, however, that are not conducive to this simple method of treatment. It may be more expedient and economical, on the whole, to use the dispersion box and the spray or dry crystal blower methods of distribution.

With Other Chemicals

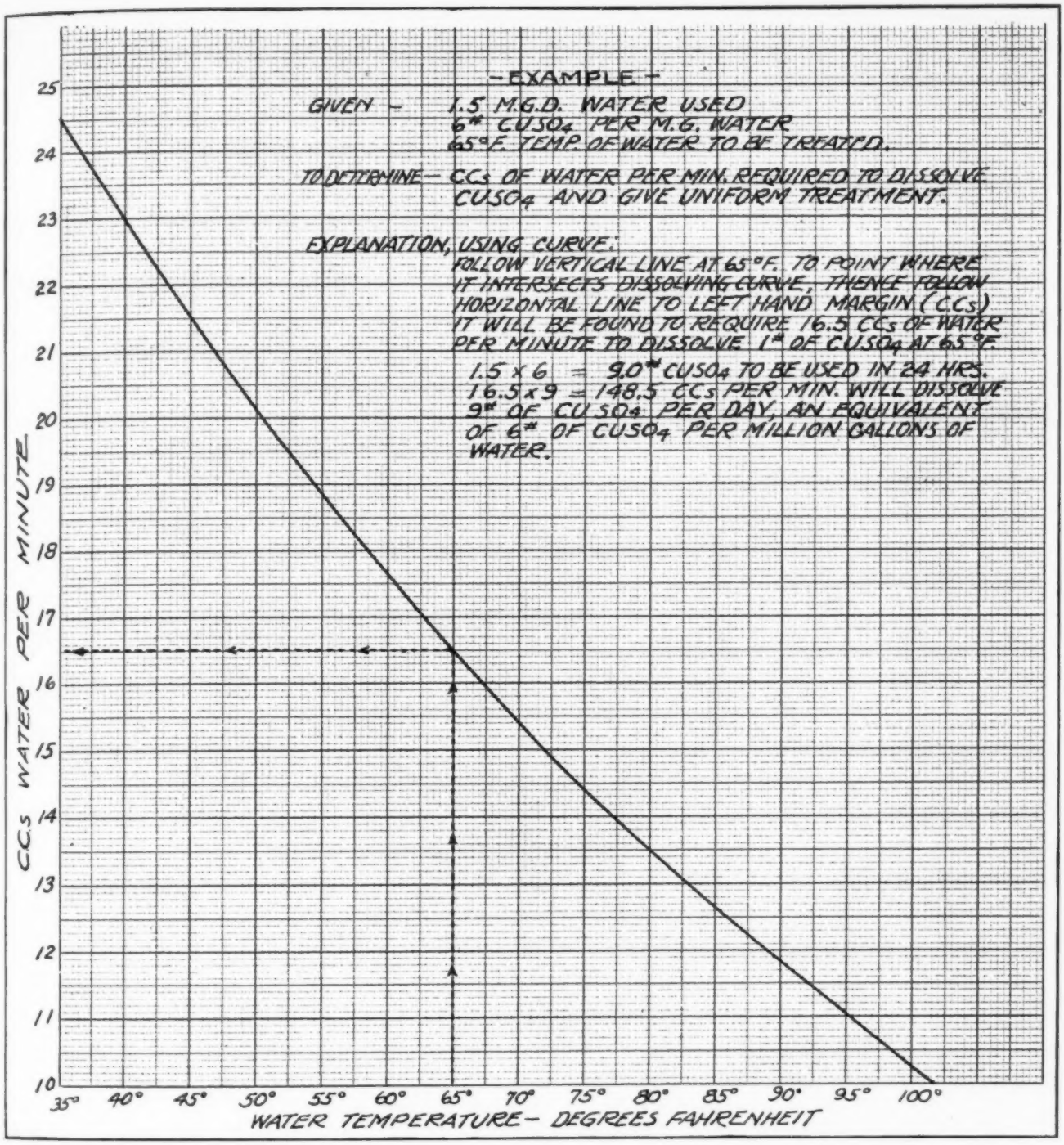
It has been frequently found necessary to apply copper sulphate continuously, at the inlet of the mixing chamber, or at other convenient points, during times when micro-organisms are most abundant and are not entirely reduced by reservoir treatment alone. This may be accomplished by making a concentrated solution in a paraffine-lined wooden barrel, or other acid resisting container. An orifice or stopcock drains the solution from the container into the water to be treated, necessitating recharging at frequent intervals. This type of equipment is usually a makeshift and is not conducive to uniform and economical feeding.

We at Atlanta have endeavored to make our copper feeding equipment permanent—(see illustrations). These feeders are of the solution type, and are mounted out-of-doors at the influent and effluent ends of the mixing chamber. A standard 3 foot length of 24 inch diameter terra cotta pipe is set in 4 inches of concrete grout, with the bell at the top. The lower 18 inches of this pipe is filled with sand and sealed with about 3 inches of concrete. Into the top 18 inches is fitted a circular lead container which rests upon the concrete slab. A heavy walled lead pipe of 1 in. diameter leads from the bottom of the circular lead container down and through the side of the terra cotta pipe and

One of the Atlanta Copper Sulphate Feeders
(Modified Shelton Design) Constructed by Atlanta Water Department (See detail sketch)



The Atlanta Copper Sulphate Feeder in Detail
(Note simplicity of construction and dosage control by needle-valve setting for water feed only)


-EXAMPLE-
GIVEN - 1.5 M.G.D. WATER USED
6# CUSO4 PER M.G. WATER
65°F. TEMP. OF WATER TO BE TREATED.
TO DETERMINE- CCs OF WATER PER MIN. REQUIRED TO DISSOLVE CUSO4 AND GIVE UNIFORM TREATMENT.
EXPLANATION, USING CURVE:
FOLLOW VERTICAL LINE AT 65°F. TO POINT WHERE IT INTERSECTS DISSOLVING CURVE, THENCE FOLLOW HORIZONTAL LINE TO LEFT HAND MARGIN (CCs) IT WILL BE FOUND TO REQUIRE 16.5 CCs OF WATER PER MINUTE TO DISSOLVE 1# OF CUSO4 AT 65°F.
1.5 x 6 = 9.0# CUSO4 TO BE USED IN 24 HRS.
16.5 x 9 = 148.5 CCs PER MIN. WILL DISSOLVE 9# OF CUSO4 PER DAY, AN EQUIVALENT OF 6# OF CUSO4 PER MILLION GALLONS OF WATER.
CCs WATER PER MINUTE
25
24
23
22
21
20
19
18
17
16
15
14
13
12
11
10
35° 40° 45° 50° 55° 60° 65° 70° 75° 80° 85° 90° 95° 100°
WATER TEMPERATURE- DEGREES FAHRENHEIT

Courtesy—Tennessee Copper Co., Copper Hill, Tennessee

From This Graph Is Determined the Rate of Water Feed Required to Dissolve and Feed the Desired Dosage of Copper Use of the graph in arriving at the correct water rate setting per pound of copper sulphate required per hour is explained on the graph. The solution discharged is saturated, varying in concentration with the temperature of feed water, consequently the need for water temperature readings and use of this helpful graph.

to the point of application. A small, circular, perforated lead baffle fits over the inlet to the feed line to prevent stoppage. Dry crystals (random size) are placed in the circular lead dissolving pot. Water from a ⅜-in. copper line flows through a pressure reducing valve, thence through a small orifice, where it drips into the copper crystals, forming a uniform saturated solution of copper sulphate. This flows out at the bottom of the pot. The quantity of copper sulphate is readily controlled by a calibrated needle valve, which admits the water through the orifice into the bed of copper crystals. The water flow is set in accordance with the temperature-solubility curve, here reproduced, so as to secure the desired rate of copper application.

An attractive metal mushroom-type cover protects the equipment from the elements. This apparatus is inexpensive to build, easy to operate, and is subject to accurate calibration to give the required dose. Its neat design makes it an asset to any plant. It will operate without adjustment for days, still maintaining the original setting and feed rate. One decided advantage, over many other types of home-made equipment, is that the control involves the feeding of nothing more difficult than pure water through the small orifice, rather than a corrosive or encrusting solution, as is usually the case with solution feed equipment. Small particles of foreign matter or undissolved crystals soon obstruct most any type of restricted orifice when employed in feeding copper sulphate solution.

Dissolving Water at Constant Temperature

It becomes apparent, from a study of the accompanying chart, employed in arriving at the flow rate for the dissolving water

stream in dosage manipulation, that if the supply of dissolving water could be produced at a constant temperature, or at a temperature which fluctuates but slightly, a worth while advantage accrues.

To attain such a desideratum, or an approach thereto, consideration of a scheme for producing a supply of water at ground water temperature, as developed in the Atlanta Water Department, is suggested. This scheme is an invention of Mr. William Rapp, Atlanta's late Superintendent of Construction and Distribution, which was first described and illustrated in detail in "Water Works and Sewerage."

In brief, the scheme consists of employing an existing well, or putting a new 4 inch well down into ground water to such a depth that a 5 foot long coil, made up of pure copper tubing, may be submerged in the ground water for the most part, if not completely so. The inlet (upper end) of the coil connects to a drop pipe from the city supply main. The outlet (lower end) discharges through a ⅜ in. riser of copper tube, running up alongside of or through the center of the coil. The city water supply will be cooled or warmed, as the case may be, to the rather constant temperature of the ground water which is constantly moving across non-pumped wells, or upwards through pumped wells. By this method, in Atlanta, water has been cooled and kept flowing under main pressure at 66 deg. F., although the city water entering Mr. Rapp's "ground cooler" has been as high as 87 deg. F. during the summer period.

In connection with the production and displacement of a saturated solution of a chemical in dosage control, the matter of *what* temperature is not the important consideration, but rather the production of a fairly *constant* temperature feed water, and knowledge of the concentration of saturated solution at such temperature. Naturally, some thought must at the same time be given to insulation of the dissolving pot in order that the chosen water temperature may be maintained fairly constant while passing through the saturation-pot.

We realize that there are other methods of producing water at thermostatically controlled temperatures, with constancy, but we rather enjoy the idea of letting Nature contribute its bit through use of the economical and fool-proof equipment and scheme of the eminently practical Mr. Rapp.

Effect of Copper Sulphate on Organisms

Microscopic examinations and threshold odor tests are important in attaining efficient application and control of copper sulphate treatment. There is not one fixed dosage of copper sulphate suitable for any one condition; but, generally speaking, an application of 2 to 10 pounds per million gallons of water will be found effective in the elimination of the most objectionable microscopic growths. And, continuous application of small doses usually gives good results in bringing about practical elimination of most of the more common and troublesome types. A twelve month program of application, on larger reservoirs, and early season control in mixing chamber, etc., makes possible the use of small applications and effects a maximum of organism reduction without taste and odor formation.

Copper sulphate should be used to *prevent* microscopic growths in a water purification system, rather than to kill the organisms after they have grown to large numbers. A small quantity of copper sulphate applied continuously during the growing season, or certainly at frequent intervals, will keep growths at a minimum and will also prove over-all a more economical procedure.

When it is convenient to lower the water line in a water treatment plant or dewater it entirely, algae adhering to the walls may be eliminated by spraying or washing the exposed area with a 5% solution of copper sulphate.

Filter Bed Clogging

A number of water plant operators have experienced a partial if not an almost complete clogging of filter beds by micro-organisms. Greatest relief has been had when copper sulphate treatment was begun and continuously applied during the propagation season. Diatoma and synedra are usually the chief offenders and may be killed, or reduced, by using approximately 2 to 6 pounds of $CuSO_4$ per million gallons of water.

At Spartanburg (1), Marquis describes a situation where a new plant was put into service, after the supply reservoir had been idle a long time, permitting a luxuriant growth of algae. Rapid sand filters became so densely matted with algae that filter runs were reduced to 1½ hours between washings. The wash water consumption reached 25% of the total pumpage and the city was on the verge of a water shortage. Copper sulphate was immediately applied to an area 1,000 feet in front of the raw-water intake by dragging copper sulphate crystals in a burlap bag fastened behind a boat. Approximately 13 pounds of $CuSO_4$ were used for each million gallons of water. This treatment was continued daily until relief was apparent at the filter plant. After the initial emergency was over, copper sulphate was regularly applied by solution feed at the inlet of the mixing chamber. Dosages ranged from 2 to 13 pounds per million gallons. This same procedure has been followed yearly since, without similar reoccurrence.

A few years ago, Atlanta experienced the formation of troublesome sheets of microorganisms, in the impounding reservoirs, which reduced filter runs from 120 hours to 20 hours between washings. Taste and odor complaints ensued. This was rectified by monthly applications of $CuSO_4$, at the edge of the reservoir and also continuous application at the inlet of the mixing chamber, when the raw water turbidity drops below 25 p.p.m. At such times 2 to 7 pounds of $CuSO_4$ per million gallons of water are used to combat these growths, with satisfactory results. The continuous application of 4 pounds of copper sulphate per million at Atlanta, during periods of algae propagation, has materially reduced tastes and odors, chlorine consuming substances, and coagulant used. This results in a more palatable drinking water, at a reasonable cost, and filter runs at Atlanta are almost directly proportionl to the concentration of micro-organisms in the *applied* water.

The Question of Copper on Health

It is believed that the judicious use of copper sulphate to prevent and kill algae in water purification systems is not harmful to health and is sanctioned by practically every State Department of Health in the U. S. A. (2).

Reliable authorities (3) have more recently proven that doses of $CuSO_4$, as high as 100 pounds per million gallons, could be used with alum coagulation, if the pH of the water is above 6.3, without danger of copper passing the filters.

Negus (4) states that it is necessary for about 30 p.p.m. of copper ions (120 p.p.m. of copper sulphate) to be present in water to stain white plumbing fixtures. He further states that research and experience have proven that a copper limit of 0.2 p.p.m. (about one-half the amount present in cow's milk) is much lower than necessary. Conservatively, five times this amount (1.0 p.p.m.) would still be considered safe.

Uses Other than as an Algaecide

Copper sulphate is said to increase the toxicity of ammonia-chlorine sterilization under certain conditions.

At Indianapolis the combination of pre-chlorination and copper sulphate has proved highly effective.

Copper sulphate has been successfully used, with aluminum sulphate, to control algae growths on the vertical side walls of reservoirs and basins. Eight to 16 pounds per million gallons of copper sulphate proved satisfactory in this undertaking (5).

Copper salts are highly effective coagulants and Damn and Bock (6) report that when milk waste was treated with two pounds of copper sulphate to between 225 and 275 cu. ft. of waste a good floc was produced. A clear effluent was obtained by passing the flocculated material through a magnetite filter. The precipitated sludge is said to have possibilities as a fertilizer.

Conclusions

Fundamentally, all structures entering into the storage, purification, and distribution of water supplies should be maintained in a high degree of cleanliness, free from slimes, growths and deposits. Treatment with copper sulphate continuously, where possible, or intermittently where absolutely necessary due to local conditions, accomplishes this.

Experience in many plants shows that copper sulphate contributes to better operation not only through algae control but betterment of plant operation by its effect upon floc, filter condition, and appearance of finished water.

During 40 years of usage copper sulphate has demonstrated its indispensability in water treatment and deserves regard as one of our most important water purification chemicals.

Like the famed "ounce of prevention," a pound of copper effectively timed and placed is worth more than "tons" of regrets and explanations.

References

1. J. K. Marquis—"$CuSO_4$ Treatment at Spartanburg, S. C.," J. SE Section A.W.W.A., 1933.
2. George A. Spalding—Round Table W. W. Eng., May 1933. 1933.
3. C. J. Brockman—"Adsorption of $CuSO_4$ by Aluminum Floc," J. SE Sec. A.W.W.A., 1933. Dr. Parker, SC.B., W. W. Eng., June 28, 1932.
4. Sidney S. Negus—"Physiological Aspects of Mineral Salts in Public Water Supply," A.W.W.A., Jour. Feb., 1933.
5. G. F. Gilkison—"Use of $CuSO_4$ in Kansas City Reservoir." Public Wks., Mch., 1921.
6. Damn & Bock—Pub. Health Abs., Apr. 1937.

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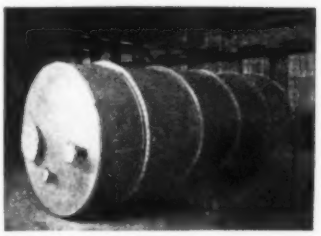

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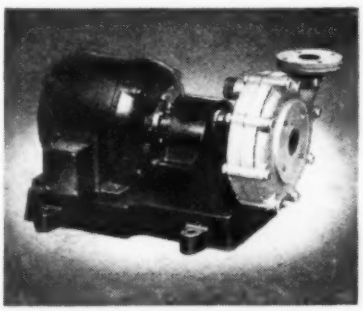

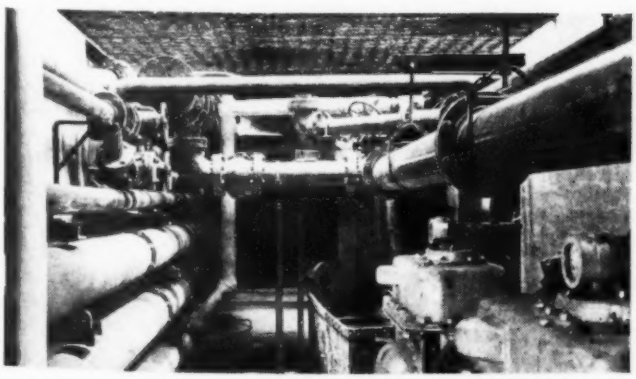

In this sewage installation, Ace hard rubber lined pipe, valves, fittings; Ace solid hard rubber pipe and fittings, and an Ace rubber-lined tank were used for maximum corrosion resistance.

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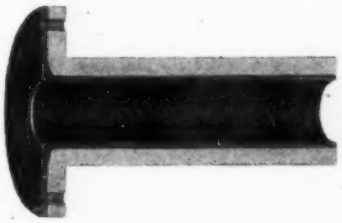

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64 PAGE CATALOG

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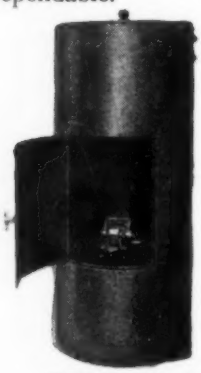

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CONVENIENCE

W&T is represented in each of the 35 cities listed above by chlorination specialists to assist in selecting proper equipment for any chlorination problem. Their recommendations involve no obligation.

Current literature on the many phases of chlorination, ammoniation and dry chemical feeding is available on request.

THE CHLORINE DEMAND DETERMINATION*

Ed. Note: Although the technic given here was prepared by the AWWA Committee for Chlorination Control of Water, it is suitable for use on sewage providing the operator makes a few minor changes in detail. For example, the size of the sample may have to be varied; the incremental additions of chlorine reagent water will generally be different; the contact time for the determination of the chlorine demand of sewage is standardized at 15 minutes; and 5 portions are usually sufficient.

For sewage, some estimate of the increments of chlorine reagent to be used may be obtained by determining the immediate chlorine demand (I.C.D.), using a spot plate with 1 ml. portions of the sample and 3 drops of ortho-tolidine for ascertaining the residual chlorine, allowing 30 to 45 sec. for color development. (The minimum readable color in the spot plate is between 0.05 and 0.1 pp.)

The I.C.D. may be expected to be from 60 to 80 per cent of the total (15 min.) chlorine demand.

For plant control or field work, the spot plate technic for the determination of residual chlorine is simple and sufficiently accurate, and it obviates the need for standards and apparatus.

THE chlorine demand method presented is designed to lend itself to the determination of the so-called immediate demand or to any other demand using longer contact time or larger chlorine dosages. It can be used, for instance, in the determination of the break-point. It specifies that chlorine water be used as the source of chlorine when liquid chlorine is used in operation. The alternate use of "Zonite" as the source of chlorine, given in Part B, is more convenient for use in the field or when facilities are not available for standardizing chlorine water. Generally speaking, it would appear that results with "Zonite" will be of the same order of accuracy as with chlorine water. It is suitable for use on either water or sewage.

The chlorine demand of water is the difference between the amount of chlorine applied and the amount of residual chlorine remaining at the end of the contact period. The demand for any given water varies with the amount of chlorine applied, time of contact and temperature. For comparative purposes it is imperative that all conditions be stated. The smallest amount of residual chlorine considered at all significant is 0.1 ppm. Treatment of water beyond this point usually results in a series of increasing demands. Addition of that amount which will destroy the ammonia present is commonly termed "break-point" chlorination. In practice, the ratio of chlorine to ammonia nitrogen (NH_3—N) at the break-point is seldom less than 10 to 1 and may reach 25 or more to 1.

All chlorine demands may be determined by following the same general procedure.

Part A. Method for Laboratory Use

Sec. 1—Reagents

1.1. *Ortho-tolidine Reagent—Method 1:* Dissolve 1.35 g. ortho-tolidine dihydrochloride in 500 ml. distilled water. Add this solution, with constant stirring, to 500 ml. dilute hydrochloric acid made by mixing 350 ml. distilled water and 150 ml. concentrated hydrochloric acid (sp.gr. 1.18-1.19).

If it is necessary to use orthotolidine, this reagent should be obtained in the purest possible condition. Even so, ortho-tolidine contains more impurities than the ortho-tolidine dihydrochloride and is therefore less satisfactory.

1.2. *Ortho-tolidine Reagent—Method 2:* Grind 1 g. ortho-tolidine in a mortar with 5 ml. dilute hydrochloric acid, made by adding 150 ml. concentrated hydrochloric acid (sp.gr. 1.18-1.19) to 350 ml. distilled water.

Add 150-200 ml. of distilled water. The ortho-tolidine goes into solution immediately.

Transfer the solution to a 1.0-liter graduate and add the balance (495 ml.) of the dilute acid (A-1.2) and make up to 1.0 liter with distilled water.

1.3. *Storage of Ortho-tolidine Solution:* The ortho-tolidine solution should be: (a) stored in amber bottles or in the dark, (b) protected at all times from direct sunlight, (c) used no longer than six months, (d) kept from contact with rubber and (e) maintained at normal temperatures.

At temperatures less than 0° C., the ortho-tolidine will precipitate from solution and cannot be redissolved easily. The use of the reagent from which part of the ortho-tolidine has precipitated may lead to errors due to a deficiency of ortho-tolidine.

1.4. *Chlorine Water Solution:* A suitable solution may be obtained from the chlorinator solution hose or by bubbling chlorine gas through water obtained from the laboratory tap. Due to instability of chlorine water, it is necessary to make up fresh solutions daily under any conditions.

Standardize the chlorine solution immediately prior to the start of each test. The use of amber bottles is recommended. When hypochlorite is used in plant operation, it may be used for this test.

The strength of the chlorine water or hypochlorite solution should be such that the volume of the treated sample will not be increased by more than 5 per cent.

Standardize the chlorine solution iodometrically (Sec. C-3).

Sec. 2—Procedure

2.1. *Volume of Samples:* Measure at least ten equal portions of the water under test. Use beakers or other glass containers, such as fruit jars, of ample capacity to permit mixing the chlorine with the water. If the object of the test is simply to determine chlorine demand, the volume of the sample in each container may be no more than 100 ml. If the purpose of the test is to relate chlorine demand to bacterial removal, the effect on taste and odor or the chemical constituents of the water, samples of 500 ml. or more will be required.

2.2. *Addition of Chlorine Water:* The amount of chlorine added to the first jar should be such that no chlorine remains at the end of the contact period, especially when low demands are being studied. Add increasing amounts of chlorine to successive portions in the series. The increase between portions may be as low as 0.1 ppm. for determining low demands and up to 1.0 ppm. for exploring higher demands.

Mixing is imperative while the chlorine solution is being added to the sample.

2.3. *Contact Time:* To determine in advance the effect of chlorination, the plant contact time and temperature should be duplicated in the laboratory. For plant control, other and shorter contact periods at some controlled temperature generally can be found. During the contact period, the chlorinated samples must be protected from strong daylight.

2.4. *Examination of Samples:* At the end of the contact period, remove an equal amount from each portion and determine the residual chlorine by either the ortho-tolidine or iodimetric method.

It will aid in studying the results to plot the residual chlorine or the amount consumed.

Samples for bacteriological examination may be removed at desired intervals.

Taste and odor of the treated samples may be observed at ordinary temperatures without de-chlorination. For odor observation at elevated temperatures, samples should be de-chlorinated before heating. Choice of the de-chlorinating agent must be made with due regard to its effect on odor in the water under examination. Generally, sodium bisulfite, using only small excess, is satisfactory.

Part B. Method for Field Use

This test is designed for the measurement of the chlorine demand in the plant or the field when adequate facilities or personnel are not available to employ the more exact method (Part A). Results obtained in this test are approximations only.

Sec. 1—Reagents

1.1. Use ortho-tolidine reagent (Part A, Secs. 1.1 and 1.2).

1.2. *Chlorine Solution:* Use "Zonite." This preparation contains approximately 1 per cent available chlorine by weight. For this test its volumetric measurement is sufficiently accurate.

Sec. 2—Equipment

2.1. One medicine dropper which will deliver 20 drops per ml.
2.2. Ten 1-qt. fruit jars marked at the 500-ml. point.
2.3. Ten 2-oz. bottles marked at the 20-ml. point.
2.4. Glass stirring rod.
2.5. Glass-stemmed thermometer.
2.6 Permanent color standards prepared, as given in the article on the orthotolidine-arsenite test by Gilcreas, or commercially prepared.

*Excerpted from the AWWA Committee Report on "Control of Chlorination" and reprinted by permission of the Association.

Sec. 3—Procedure

3.1. *Measurement of Samples:* Fill each jar to the 500-ml. mark with the water under test. Record temperature at which samples are held.

3.2. *Addition of Chlorine:* While stirring constantly, add 1 drop "Zonite" to water in the first fruit jar, 2 drops to that in the second jar, 3 drops to that in the third jar, etc. Each drop of "Zonite" represents a chlorine dose of 1.0 ppm.

3.3. *Contact Time:* The contact time is a variable factor depending entirely on the results desired. Generally, the purpose of a chlorine demand test is to determine the amount of chlorine required to produce a definite residual after a definite time interval.

It is recommended, therefore, that the testing be carried out at the end of the contact period corresponding to the contact at the point of control. This may vary all the way from a few minutes to several hours. Under some circumstances it is desirable to make several chlorine determinations at different times of contact, such as 15, 30, 60 min., etc. Such a procedure will give an indication of the stability of the chlorine as related to time, which will be very useful information in plant control.

The time of contact must be recorded. During the contact period the chlorinated samples must be protected from strong daylight.

3.4. *Examination of Samples:* At the end of the contact period, remove a 20-ml. portion from each sample, place it in a 2-oz. bottle and add 1 ml. ortho-tolidine to each 20 ml. Determine residual chlorine by comparison with permanent standards.

3.5. *Calculation:* Subtract the amount of residual chlorine (in ppm.) found in examination of samples from the amount of chlorine added (in ppm.). The result will be the chlorine demand (in ppm.) of the sample at the respective dosage, contact time and temperature. Plotting the residual chlorine or the amount consumed will aid in studying the results.

Part C. Iodometric Method for Residual Chlorine

The iodometric method for the determination of residual chlorine in water can be made more precise than the ortho-tolidine method, particularly when residual chlorine is greater than 1 ppm. When the titration is made without acid, the interfering effect of nitrites and manganic and ferric salts is minimized. Generally, the starch-iodide method carried out in acid solution yields higher figures than the neutral starch-iodide titration. The acid titration therefore is preferred when interfering substances are known to be absent. Acetic acid should be used, but if not available, sulfuric acid of cp. quality may be substituted. *Hydrochloric acid should not be used.*

Colorimetric methods for the iodometric determination of chlorine are available, but are not given in this report.

Sec. 1—Reagents

1.1. *Potassium Iodide Solution (KI):* Dissolve 75 g. cp. potassium iodide (free from iodine and iodate) in 1.0 liter freshly boiled and cooled distilled water. If difficulty is experienced with stability of the solution, the solid substance may be used directly, since it is very soluble.

1.2. *0.01N Sodium Thiosulfate:* Stability of 0.01*N* thiosulfate is improved if it is prepared by diluting an aged 0.1*N* solution (made as directed in Part C, Sec. 1.2.1) with freshly boiled distilled water. Boiled water is used because bacterial action decomposes 0.01*N* thiosulfate. Trouble of this sort may be avoided by the addition of a few milliliters of chloroform. For accurate work, this solution should be standardized daily. The use of an automatic burette, of a type in which rubber does not come in contact with the solution, is advisable.

1.2.1. *0.1N Sodium Thiosulfate:* Make 0.1*N* sodium thiosulfate by dissolving at least 25 g. of the cp. salt in 1.0 liter of freshly boiled distilled water, standardizing it against potassium dichromate, after at least two weeks' storage.

1.3. *Starch Solution:* To 5 g. starch (potato, arrowroot or soluble) in a mortar, add a little cold water and grind to a thin paste. Pour into 1 liter of boiling distilled water, stir and allow to settle overnight. Use the clear supernatant. The solution should be preserved with salicylic acid (1.25 g. per liter of starch solution). The starch may also be preserved with zinc chloride added at the rate of 4 g. per liter.

1.4. *Acid Solution:* Use either (a) a solution of acetic acid, made by diluting 500 ml. glacial acetic acid to 1.0 liter with distilled water, or (b) a solution of sulfuric acid, made by adding 20 ml. of concentrated sulfuric acid (sp. gr. 1.84) to approximately 750 ml. distilled water and diluting to 1.0 liter. Add the acid to the water slowly, with constant stirring.

Sec. 2—Procedure

2.1. *Volume of Sample:* The amount of sample to be taken for titration is governed by the concentration of chlorine in the sample. It is suggested that, for quantities of residual chlorine of 1.0 ppm. or less, 1.0 liter be titrated; for residual chlorine concentrations between 1 ppm. and 10 ppm., 500 ml.; for residual chlorine concentrations above 10 ppm., proportionately less of the sample should be used. It is preferable to use sufficient sample so that not more than 20 ml. of 0.01*N* thiosulfate is required.

2.1.1. *Preparation for Titration:* If no interfering substances are present, place 10 ml. acid, or sufficient to reduce the pH to between 3.0 and 4.0, in a flask or white porcelain casserole, add 10 ml. of KI solution, pour in the sample and mix with a stirring rod. Omit the acid if interfering substances are present.

2.1.2. *Titration:* Add 0.01*N* thiosulfate from a burette until the yellow color of the liberated iodine is almost discharged. When liter samples are titrated, add 5 ml. starch, reducing this amount with smaller samples. The amount of starch used should be sufficient to give a deep blue color. After the addition of starch, titrate carefully, but rapidly, to the end-point. Long contact of iodine and starch produces a blue compound which is decolorized with difficulty.

2.2. *Calculation of Results:* The number of milliliters of 0.01*N* thiosulfate used in 1.0 liter of sample multiplied by 0.3546 equals the chlorine present, in parts per million.

Sec. 3—Standardization of Chlorine Solution

3.1. *Volume of Sample:* In standardizing the chlorine solution for temporary standards, the volume taken for titration should be such as to require not more than 20 ml. 0.01*N* thiosulfate.

In standardizing chlorine water solution (Part A, Section 1.4) for determining chlorine demand, the volume taken for titration should be such as to require not more than 20 ml. of 0.1*N* thiosulfate.

3.1.1. *Preparation for Titration:* Proceed as in Part B, Sec. 2.1.1. Distilled water may be added at the discretion of the analyst if a larger volume is preferred for titration.

3.1.2. *Titration:* Titrate the chlorine solution as in Part B, Sec. 2.1.2.

3.2. *Calculation of Results:* In standardizing chlorine solution for temporary standards:

$$\text{mg./ml. chlorine} = \frac{\text{ml. 0.01 } N \text{ thiosulfate} \times 0.3546}{\text{ml. sample titrated}}$$

In standardizing chlorine water solution for determining chlorine demand:

$$\text{mg./ml. chlorine} = \frac{\text{ml. 0.1 } N \text{ thiosulfate} \times 3.546}{\text{ml. sample titrated}}$$

References

AWWA Committee: Report on Control of Chlorination. *JAWWA* 35, 1315 (1943).

KEEPING LIME SOLUTION LINES CLEAN

To retard building up of incrustation and depoits, which choke pipe lines conveying lime suspension, the following has been found very helpful at the Richmond, Va., Filter Plant. A program clock at 4 hour intervals opens and closes a flushing valve connected into the line at the inlet end. The line is thereby automatically scoured free of the loosely deposited carbonate and hydrate 6 times daily.

If a scale gradually builds up which the Richmond scheme of water scour will not remove, chlorine water from a chlorinator can be run through for the required period to dissolve or loosen the scale. Chlorine water applied to lime carbonate reacts to produce hypochlorous acid, carbon dioxide and calcium chloride as indicated by the reaction.

$$CaCO_3 + H_2O + 4\,Cl = 2\,HO\,Cl + CaCl_2 + CO_2$$

Pouring chlorine water slowly through a column of ground limestone or marble will demonstrate the reaction and is a well known method of preparing hypochlorous acid. The solution resulting, because of the weakness of the hypochlorous acid produced, has a pH of 8.0 to 8.2.

After the carbonate is all converted, chlorine water will naturally attack metallic solution lines. Lines of rubber hose with jointing made by butting hose ends together *inside* of a short sleeve of larger size hose, or a short section of pipe, provides a very satisfactory all purpose chemical feed line.

IMPROVED RESIDUAL CHLORINE TEST*

The Ortho-tolidine: Arsenite (OTA) Method for Testing Water for True, False and Chloramine Residuals

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THE reaction of free available chlorine with ortho-tolidine is practically an instantaneous one, requiring less than fifteen seconds for development of the color; thus the appearance of an immediate or flash color may be used as a measure of free chlorine present.

This flash color may be quantitatively estimated if a weak reducing agent such as sodium arsenite is added to the sample approximately fifteen seconds after the addition of the ortho-tolidine. (Step A) The sodium arsenite neutralizes all combined chlorine before it can react with the ortho-tolidine reagent. The flash color is not affected by the sodium arsenite reagent, but remains unchanged and may be read by comparison with permanent standards at any time within five minutes.[1]

When sodium arsenite is added first to the sample, (step B) all chlorine, both free and combined, is neutralized. Ortho-tolidine reagent added subsequently reacts with manganese and other interfering substances present, which are not affected by the sodium arsenite reagent, but which produce a color with ortho-tolidine similar to that indicating chlorine. This color may likewise be measured by comparison with chlorine standards.

Using readings obtained from these two procedures (steps A and B) and that of the standard ortho-tolidine test, it is possible to measure free available chlorine, combined chlorine, and ortho-tolidine color caused by interfering substances. The portion of sample to which ortho-tolidine reagent is added first, followed by the arsenite reagent, (step A) yields a color representing both free available chlorine and any interfering substances present. In the portion to which arsenite is added first, followed by ortho-tolidine reagent, (step B) the color produced represents interfering substances only. Thus, free available chlorine may be determined by subtracting the second value from the first. The false residual chlorine value is the second reading, (obtained in step B). If the reading of the first step is subtracted from that of the standard ortho-tolidine test for total chlorine, the residual chlorine value corresponding to combined chlorine or chloramine is obtained.

The determination of the free available chlorine, the combined chlorine, and the interfering substances may be facilitated by use of a technic employing the double-bottle method. Under routine operating conditions, however, it may be possible to dispense with this procedure and use only those determinations that are of most help in the control of chlorination in each particular plant. In any case the OTA test does not outmode present standard equipment for determining residual chlorine.

Reagents[2]

Ortho-tolidine reagent—method 1. Dissolve 1.35 grams ortho-tolidine dihydrochloride in 500 ml. distilled water. Add this solution, with constant stirring, to 500 ml. dilute hydrochloric acid made by mixing 350 ml. distilled water and 150 ml. concentrated hydrochloric acid (sp. gr. 1.18-1.19).

Ortho-tolidine reagent—method 2. Grind 1 gram ortho-tolidine in a mortar with 5 ml. dilute hydrochloric acid made by adding 150 ml. concentrated hydrochloric acid (sp. gr. 1.18-1.19) to 350 ml. distilled water. Add 150-200 ml. of distilled water. The ortho-tolidine goes into solution immediately. Transfer the solution to a 1-liter graduate and add the balance (495 ml.) of the dilute acid and make up to 1 liter with distilled water.

Storage of ortho-tolidine reagent. Dispense this reagent in two-ounce French square brown glass bottles. Store in dark when not in use, and avoid exposure to low temperatures. The solution should be clear and colorless; discard it if it becomes either colored or turbid. Certain types of rubber stoppers are attacked by the strong acid and yield reducing substances that cause the reagent to deteriorate.

At temperatures below 0° C. the ortho-tolidine will precipitate from solution and will not redissolve readily. Any reagent containing a precipitate should not be used.

Sodium arsenite reagent. Dissolve 0.5 grams of sodium arsenite in distilled water and make up to 100 ml. Store in any sized bottle but use in a properly labeled 2-oz. French square reagent bottle with a screw cap medicine dropper.

*This article is cited from material prepared by the author for use, in New York State, in connection with the OTA Test.

Permanent Chlorine Standards[3]

0.5 *M phosphate buffer solution.* Allow a sufficient quantity of disodium hydrogen ortho-phosphate dodecahydrate ($Na_2HPO_4 \cdot 12H_2O$) to dry over a saturated solution of potassium carbonate ($K_2CO_3 \cdot 2H_2O$) until the weight remains constant for one week, and weigh as $Na_2HPO_4 \cdot 2H_2O$. If disodium hydrogen ortho-phosphate anhydrate (Na_2HPO_4) is used, dry at 110° C. and cool in a desiccator.

Dissolve 28.66 grams disodium hydrogen ortho-phosphate dihydrate ($Na_2HPO_4 2H_2O$) (or 22.86 grams Na_2HPO_4) and 46.14 grams potassium dihydrogen ortho-phosphate anhydrate (KH_2PO_4) in 1 liter of distilled water. Let stand for at least several days before using, to allow time for formation of any precipitate, which must be filtered out prior to use.

0.1 *M phosphate buffer solution* (pH 6.45). Filter solution as prepared above and dilute 200 ml. to 1 liter with distilled water.

Strong chromate-dichromate solution. Dissolve 1.55 grams potassium dichromate ($K_2Cr_2O_7$) and 4.65 grams potassium chromate (K_2CrO_4) in 0.1 M phosphate buffer and dilute to 1 liter with the 0.1 M phosphate buffer. This solution represents 10.0 ppm. chlorine when viewed through a 240-300 mm. depth.

Dilute chromate-dichromate solution. Dissolve 0.155 grams potassium dichromate ($K_2Cr_2O_7$) and 0.465 grams potassium chromate (K_2CrO_4) in 0.1 M phosphate buffer and dilute to 1 liter with the O.1 M phosphate buffer. This solution may also be prepared by diluting 100 ml. strong chromate-dichromate solution to 1 liter with 0.1 M phosphate buffer. The solution represents 1 ppm. chlorine.

Preparation of standards. Measure the volumes of dilute chromate-dichromate solution given in Table I into clean two-ounce French square bottles of clear glass and dilute to exactly 50 ml. with the 0.1 M phosphate buffer solution.

TABLE I

Chlorine Standards—Modified Scott Formula 0.05-1.0 ppm.

Chlorine Standards ppm.	Dilute chromate-dichromate solution ml.	Chlorine Standards ppm.	Dilute chromate-dichromate solution ml.
0.05	2.5	0.40	20.0
0.10	5.0	0.50	25.0
0.15	7.5	0.60	30.0
0.20	10.0	0.70	35.0
0.25	12.5	0.80	40.0
0.30	15.0	1.00	50.0

The standards should be protected from contamination by dust by keeping the bottle closures tightly screwed down or by sealing the closure with collodion or paraffin. The standards must be stored in the dark and renewed if any trace of turbidity appears in them.

Apparatus

1. Two-ounce French square bottles.
2. Medicine dropper caps to fit French square reagent bottles.
3. Commercial test kits may be used.

Procedure for Ortho-tolidine-Arsenite (OTA) Test

1. Select three two-ounce French square bottles; mark one A, one B, and one OT.
2. Add 50 ml. of a sample of chlorinated water to each of the three bottles.
3. Unscrew, and fill droppers of both reagent bottles, one with ortho-tolidine, the other with arsenite.
4. Hold samples for the required period of contact, usually ten minutes, unless a period longer than that has elapsed between the application of the chlorine and collection of the sample.

   All operations noted in paragraph 5 should be carried out as rapidly as possible. For each bottle, the addition of reagents and mixing should be completed within ten to fifteen seconds.
5. Add a dropperful of ortho-tolidine to bottle marked OT. Mix by one quick shake.
   Add a dropperful of ortho-tolidine to bottle marked A; mix.
   Add a dropperful of arsenite; mix.

Add a dropperful of arsenite to bottle marked B; mix. Add a dropperful of ortho-tolidine; mix.

6. If the sample is cold, warm portion OT to 68° F. in a bath of warm water or by holding in the hand. Portions A and B do not require warming since the rate of color development is independent of the temperature.
7. Free available residual chlorine. Three minutes after addition of the ortho-tolidine, read portions A and B in comparison with color standards. Subtract the reading for B from that for A; the difference is the free available chlorine present.
8. False residual. Read portion B alone in comparison with color standards four minutes after addition of ortho-tolidine. If sample is colored or turbid use an untreated sample blank as in the standard ortho-tolidine test. This reading is due to interfering agents only.
9. Standard OT test. Read OT portion at four and one-half minutes in the usual manner.
10. Combined chlorine. Five minutes after addition of ortho-tolidine, read portions OT and A in comparison with color standards. Subtract the reading for A from that for OT; the difference is the combined chlorine (chloramine). If this reading exceeds 0.05 ppm., read again at fifteen minutes to allow for the slower development of color due to combined chlorine. Read portion OT again just prior to the fifteen-minute combined chlorine reading.
11. The following table gives the times and relations between the various readings.

	Read at*	
A minus B	3 minutes	= free chlorine
B	4 minutes	= false residual
OT	4½ & 14½ minutes	= total chlorine & false residual
OT minus A	5 & 15 minutes	= combined chlorine

Procedure for Ortho-tolidine-Arsenite Test Using Commercial Test Kits

Four cells are necessary to perform the complete test. If only two are available, the test may be divided into two parts, the A and B portions and then the OT and A portions repeated. A large sample of water should be collected and both parts of the test carried out on the same sample. The procedure and the amounts of reagents are the same as for the test using 50-ml. portions in French square bottles. Because of the higher acid concentration present when the reagent is added to 15-ml. portions, it is not necessary to wait fifteen minutes for full color development due to combined chlorine; five minutes is sufficient.

*The combined chlorine reading must be made exactly at five minutes and at fifteen minutes because of the changing color. For the same reason it is essential to read B just after A minus B and to read OT just before OT minus A. The timing for the other readings is suggested for convenience.

Limitations

Certain limitations in the technic are mentioned by Hallinan in his original publication of the ortho-tolidine-arsenite test.[1] In the test for chloramine, owing possibly to the incomplete reaction between arsenite and chloramine, the results may be 3.5 per cent low, an appreciable but not significant error. Products of the reaction of chlorine with organic matter, particularly amines, may also be included in the chloramine reading; thus, differentiation of chloramine from other forms of combined chlorine cannot be made.

The use of two reagents and the speed required in their addition have not been found to complicate the technic. In general practice, the technic has proven effective in the detection of interfering substances in an amount not greater than the equivalent of 1 ppm. of chlorine. As with all colorimetric methods, the precision of the test is governed by the limitations of color comparison, particularly in ranges of high color intensity. The test is effective in differentiating free available chlorine from that combined with ammonia or other forms of nitrogenous organic matter.

Results of Application of the OTA Test

The test has been used routinely for a period of one month by the operators of 23 water supplies in New York State.[2] Of 659 days represented by the readings of these 23 supplies, a false residual was found on 102 days; and on 84 days the chloramine residual was 0.3 ppm. or less, with no active chlorine present. Three of the cooperating water supply operators reported definite false residual values repeatedly; and from these supplies samples of the water examined subsequently were found to contain significant amounts of manganese.

The results of this study indicated that the OTA test provides a quantitative estimation of the type of residual maintained in any given water, whether free available chlorine or the less bactericidal combined chlorine, and that it also provides a precise reading for the determination of substances which interfere with the ordinary ortho-tolidine technic. Thus, it has a definite application in the control of chlorination of water, particularly for those supplies in which such interfering substances may be continuously or intermittently present.

References

1. Hallinan, F. J. Tests for active residual chlorine and chloramine in water. *Jour. Amer. Water Works Assoc.*, **36**, 296-302 (1944).
2. Committee on Control of Chlorination. American Water Works Ass'n Report, 1943. *Jour. Amer. Water Works Assoc.*, **35**, 1315-1328 (1943).
3. Gilcreas, F. W., and Hallinan, F. J. The practical use of the ortho-tolidine-arsenite test for residual chlorine. *Jour. Amer. Water Works Assoc.*, **36**, 1343-1348 (1944).

CONCRETE

Quantities of Materials for One Cu. Yd. Rammed Concrete Based on Barrel of 3.8 Cu. Ft.

Proportions	Using ¾-in. Gravel and Under, Sand Screened Out: Cement Barrels	Sand Cu. Yds.	Gravel Cu. Yds.	Using 1-in. Stone and Under, Dust Screened Out: Cement Barrels	Sand Cu. Yds.	Stone Cu. Yds.
1:1½:3	1.71	0.39	0.78	1.85	0.42	0.84
1:2 :4	1.34	0.41	0.81	1.46	0.44	0.89
1:2 :5	1.17	0.36	0.89	1.27	0.39	0.97
1:2½:5	1.10	0.42	0.83	1.19	0.46	0.91
1:3 :5	1.03	0.47	0.78	1.11	0.51	0.85
1:3 :6	0.92	0.42	0.84	1.01	0.46	0.92
1:3½:6	0.88	0.46	0.80	0.95	0.50	0.87
1:3 :7	0.84	0.38	0.89	0.91	0.42	0.97
1:3½:7	0.85	0.43	0.85	0.87	0.47	0.93
1:4 :7	0.77	0.47	0.81	0.83	0.51	0.89
1:4 :8	0.71	0.43	0.86	0.77	0.47	0.93

Proportions	Using 2½-in. Stone, Most Small Stone Screened Out: Cement Barrels	Sand Cu. Yds.	Stone Cu. Yds.	Using 2½-in. Stone and Under, Dust Screened Out: Cement Barrels	Sand Cu. Yds.	Stone Cu. Yds.
1:1½:3	1.96	0.45	0.89	1.90	0.43	0.87
1:2 :4	1.53	0.47	0.93	1.48	0.45	0.90
1:2 :5	1.33	0.39	1.03	1.29	0.39	0.98
1:2½:5	1.26	0.48	0.95	1.21	0.46	0.91
1:3 :5	1.17	0.54	0.89	1.14	0.52	0.87
1:3 :6	1.06	0.48	0.97	1.02	0.47	0.93
1:3½:6	1.00	0.53	0.92	0.97	0.51	0.89
1:3 :7	0.94	0.42	1.05	0.92	0.42	0.98
1:3½:7	0.91	0.49	0.98	0.89	0.47	0.95
1:4 :7	0.87	0.53	0.93	0.84	0.51	0.90
1:4 :8	0.81	0.49	0.98	0.78	0.48	0.95

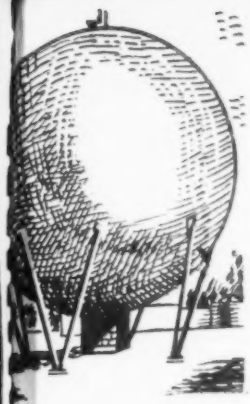
Figure about 1¼ cu. yds. of pit run gravel to make one cubic yard of rammed concrete.

Variations in fineness of the sand and the compacting quality of the concrete may affect the above quantities 5 to 10% in either direction.

Weights of Cement, Crushed Stone, Gravel and Sand

Material	Weight per Cu. Yd.	Material	Weight per Cu. Yd.
Bank sand	2,500	Screenings	2,500
Torpedo sand	3,000	Gravel	3,000
Crushed stone	2,500	Roofing Gravel	3,000

Portland cement weighs 94 lbs. per bag, or 376 lbs. per bbl. One bag is equivalent to 1 cu. ft.



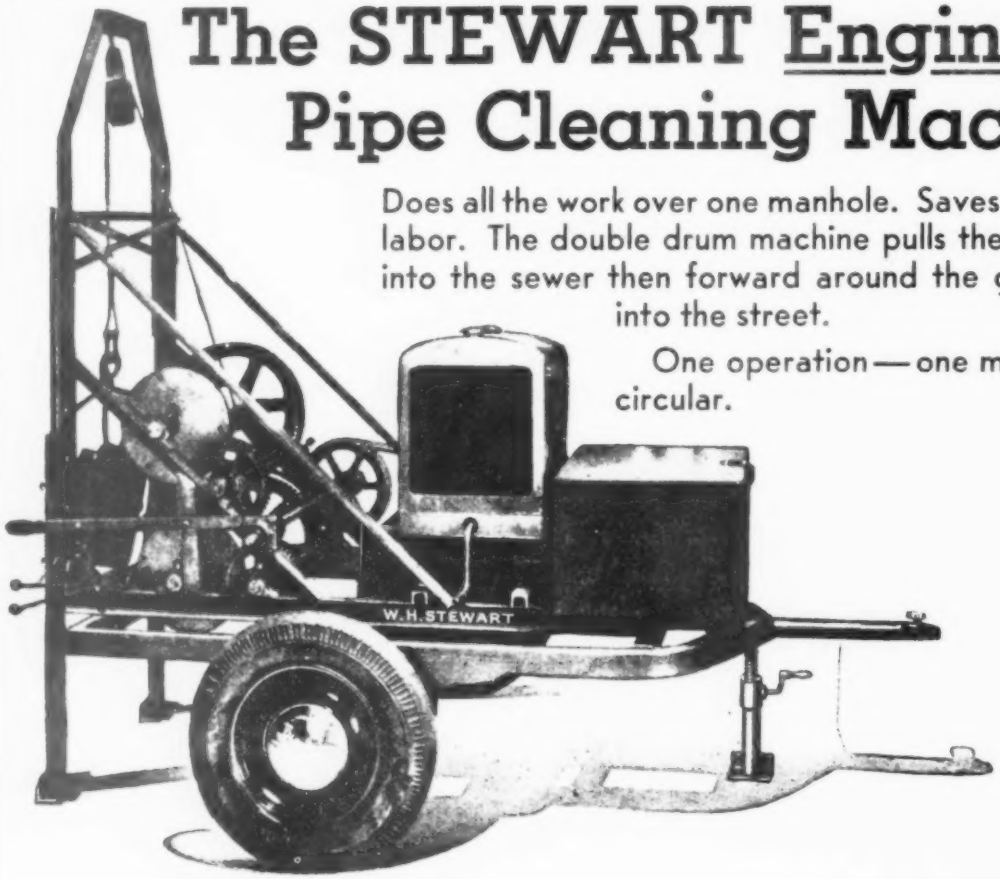
PART 7

SEWAGE TREATMENT

WATER & SEWAGE WORKS

REFERENCE & DATA SECTION 1948

The STEWART Engine Drive Pipe Cleaning Machine



Does all the work over one manhole. Saves time, money, labor. The double drum machine pulls the bucket back into the sewer then forward around the guide and up into the street.

One operation—one man. Send for circular.

OTHER STEWART ANSWERS TO SEWER CLEANING PROBLEMS

No. 10 Stewart Jumbo Sewer Rod

No. 3 Stewart Round Sewer Rod

Stewart Guard Case Bucket

For the most efficient job. For sewers 4" to 6".

No. 1 New Style Sewer Rod

Open coupling allows wood to dry thoroughly at the rivets, preventing breakage. Selected hardwood, durable and tough, with malleable iron couplings. Slide type couplings permit couplings of rods in street and minimize wear in operation. Lengths, 2, 2½, 3, 3½, and 4 ft.

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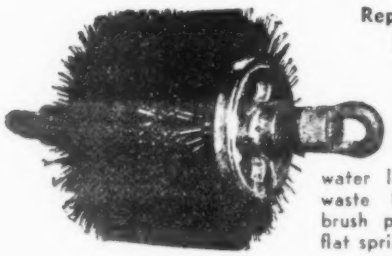

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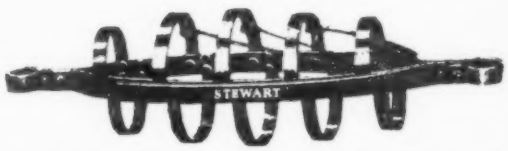

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SEWAGE TREATMENT PROCESSES AND RESULTS

By GEORGE E. SYMONS, Ph.D.
Assoc. Editor, Water & Sewage Works

BY DEFINITION there are only three processes of sewage treatment: primary treatment, secondary treatment, and disinfection. A fourth important part of any sewage treatment operation is the by-product process, solids disposal. Sewage treatment as the name implies is the treatment of liquid wastes to remove or eliminate matter which causes nuisance, and to eliminate or destroy disease producing organisms. All methods of treatment commonly called processes fall into these three basic processes of sewage disposal and their by-product process of solids disposal. Discharge of sewage into large bodies of water is sewage disposal by dilution, without treatment.

Sewage Treatment Processes

Fundamentally, sanitation or sewage treatment is for the purpose of protecting public health and preventing any and all nuisance conditions in a receiving body of water. All receiving bodies of water have a certain inherent capacity for carrying on sewage treatment processes within themselves and disposal by dilution depends entirely on that capacity. When such capacity is overtaxed, the construction of sewage treatment plants is necessitated.

The choice of the processes and methods of carrying them out is dependent on the extent to which this natural capacity is to be utilized. Any decision on the matter must include consideration of the uses to which that receiving body of water is put: water supply, fish culture, pleasure ground, commercial transport route, etc. From these considerations come not only the choice of the basic treatment processes to be used, primary, secondary or disinfection, but also the methods by which they will be carried out.

Primary treatment is the removal of solid particles by physical and mechanical means. Screening, grit removal, grease removal, and sedimentation are included in this process. Important adjuncts or aids to primary treatment or methods of improving the efficiency or effectiveness of the process include chemical precipitation, flocculation, removal of finely divided solids by aeration and vacuum flotation and the removal of grease by air, or air and chlorine combined. The slow sand filter is a primary treatment process in that it is a mechanical means of straining out solid particles. It is also a secondary process in that it depends on biological activity for destroying these solids so removed. Sand disk screens are also a method of primary treatment, as are storm water settling tanks.

Secondary treatment is an oxidation process, wet burning, a process of removal of finely divided solids and colloidal matter and the stabilization and oxidation of these solids and the dissolved organic matter by means of air and the activity of living organisms. The trickling filter and the activated sludge process are the fundamental and best known methods for accomplishing secondary treatment. Important adjuncts, aids, or improvements which increase the efficiency of the trickling filter are preaeration, and recirculation of the filter effluent with higher rates of application of the liquid to the filter. Aids or improvements in the activated sludge process are the addition of chemicals (the Guggenheim Process), tapered aeration, the Hayes Process and the Mallory Process.

Disinfection signifies the destruction of pathogenic or disease carrying bacteria; it does not mean the complete killing (sterilization) of all bacteria and lower forms of life in the sewage. Disinfection is generally accomplished by the introduction of chlorine as a solution, or the introduction of a hypochlorite solution. Most modifications of the disinfection process relate to the point of application of the disinfectant, and its possible use to control odors. Where chlorine or chlorinated products are used to control odors, a by-product process is in operation for sewage treatment, *per se*, is not involved.

Solids disposal, as the name implies, included all types of solids removed in the treatment of sewage. Screenings may be buried, burned, or ground and returned to the sewage as organic solids to be removed later as sludge. Grit may be buried or, if relatively free from organic matter, it may be used as fill or as the bottom of sludge drying beds. When high in organic matter it may be burned. Grease may be buried, burned, or possibly under some conditions, recovered.

Suspended matter removed by sedimentation and handled thereafter as a sludge may be barged to sea; dewatered, dried and burned; digested (with or without gas recovery) and dried on open or covered sand drying beds; chemically and mechanically dewatered, dried and burned; or dried and sold as fertilizer. Elutriation of such sludge is for the purpose of aiding in dewatering before drying and burning. Sludge from secondary treatment process methods may be subjected to the same treatment as primary sludge, i.e., digestion, drying, burning or drying for fertilizer purposes. Final disposition of solids, not sold or used as fertilizer, is as fill, usually near the plant site.

With this brief outline of sewage treatment process and the methods available in each, it is possible to examine the choice of these methods from a design standpoint, as dictated by the extent to which sewage treatment must be carried out. Often a factor to be considered in the choice of the process or of methods used may be the size available for the plant site. Still another factor besides the extent of treatment desired is the cost of installation versus the cost of operation.

Primary Sewage Treatment Process

In the matter of screenings and grit removal the design will probably depend on the preference of the consulting engineer, or on the bid price for the equipment needed. Whether the screenings are ground sub-aqueously or out of water is not finally determinable by data on the effectiveness of the two methods. Whether screenings will be hand or mechanically raked will depend on the size of plant and money available for construction.

Grit chambers are usually designed on the basis of the fundamental concept of differential sedimentation. That is, the design is related to the velocity necessary to keep organic particles in suspension while allowing inert particles to settle. Here the design is dependent on Stokes' law of sedimentation and the relation of specific gravity to velocity. Modifications in design are usually concerned with distribution of flow and control of velocity, and with methods of collecting and removing the settled grit. Washing of the grit will depend on the need for removing putrescible matter before the grit is disposed of. In general, washers of various design accomplish the desired results. Generally, too, such grit washers fail to remove garbage particles which are prevalent in grit from the larger communities.

In the removal of suspended matter by sedimentation there are several factors which affect the efficiency of the process besides the method employed for such removal. These factors include the concentration of the suspended matter (p.p.m.), the detention period (strictly speaking the flow through period), the distribution of flow (i.e., extent of short circuiting), the extent of comminution (dispersion) of the solid particles in the sewage, and, to a lesser degree, theoretically, the temperature of the liquid.

Whether circular tanks or longitudinal tanks should be used is a debatable question. Likewise the type of sludge collecting mechanism used is not of particular importance in the actual operation of the sedimentation process. Arguments concerning the preference of circular over longitudinal tanks usually revolve around the matter of distribution of flow. All efforts to improve the flow characteristics in either type of tank are to prevent short circuiting and to make the actual flow-through time approach the detention time, i.e., the theoretical time that any small portion of the sewage takes to pass through the tanks.

It is well known that changing the baffles in longitudinal and circular tanks will often improve the efficiency of sedimentation and consequently the removal of solids. Aside from such considerations the efficacy of various methods of primary sedimentation may be considered. Plain sedimentation for a period of one to two hours may be generally expected to remove from 30 to 50 per cent of the suspended matter present in the raw sewage and at the same time remove 20 to 30 per cent of the 5-day B.O.D. In highly efficient tanks with detention times of 2 to 3 hours, where the suspended matter reaches the treatment works with very little dispersion or subdivision of particles, the removal of the raw sewage suspended solids may reach 60 to 70 per cent.

Aids to Primary Sedimentation

As an aid to primary sedimentation, flocculation or the building up of particle size by slowly mixing the sewage before sedimentation, may increase the removal of suspended matter from 40 per cent to 60 per cent. A recent primary treatment method under development is the treatment of sewage with air followed by the application of a low vacuum which caused the fine air bubbles

to rise, carrying up small particles of suspended matter and grease. This method claims a removal of approximately 10 to 15 per cent more suspended matter than plain sedimentation. Grease removal with air, or air and chlorine, depends on the quality and amount of grease present and its characteristics as well as the design of the device and the amounts of air and chlorine added.

Slow sand filters will remove 90 per cent or more of suspended matter from raw sewage. Sand disk filters, in reality Reinsch-Wurl screens, covered with sand or crushed coal remove approximately 20 per cent of the suspended matter at best and less than 10 per cent when no filtering medium is used. Electrodialysis as an aid to flocculation gives results of 80 per cent removal of suspended solids and 70 per cent removal of B.O.D. in laboratory experiments.

Chemical precipitation with either aluminum salts or iron salts and lime will aid in the removal of suspended matter and the extent of the removal will depend largely on the amounts of chemicals used. By using large quantities of chemicals it is possible to remove 90 per cent of the suspended matter and 80 to 85 per cent of the 5-day B.O.D. Cost of such treatment is generally quite high and may be prohibitive unless the plant site is so small as to demand the installation of a highly effective method utilizing a small area. In general such quantities of chemicals are not used, and the extent of removal of solids is about 60 per cent with the B.O.D. removal about 50 per cent or better. By means of chemical treatment it is often possible to increase the effectiveness of the primary process thereby decreasing the load on the secondary process. Thus the need for further additions to plant capacity in the secondary treatment process may be eliminated.

Bases of Comparing Primary Sedimentation Results

Whereas the above comparative results have been given in terms of suspended solid removals, it should be remarked that this method of rating the efficiency of sedimentation processes is misleading and improper, if not antiquated. A much better and more applicable method of rating sedimentation processes is to compare the efficiencies as to removal of settleable solids. Obviously, as indicated above, the amount of suspended matter removed will depend on the concentration of suspendid solids and the particle size and density. Some solids should not be expected to settle under any conditions of time and space. Therefore the obvious comparative basis is the efficiency of removal of settleable solids. This efficiency is ascertainable only from the determination of the *settleable* solids.

Although this test is now used at a number of plants throughout the country, it has not been included in previous editions of Standard Methods for Sewage Analysis and only recently has a standard technique been proposed. Without information from this test available, it is impossible to make comparisons of different methods of primary treatment on this basis, but it is believed that flocculation as an aid to primary sedimentation will increase the removal of settleable suspended solids by 10 to 20 per cent, and effective chemical precipitation will, of course, extend this removal. Aeration and vacuation as recently proposed is apparently as effective as plain flocculation in the removal of suspended matter and also seems to offer additional advantages.

Secondary Treatment Process

In the matter of secondary treatment it is quite generally accepted that trickling filters and activated sludge can be designed to produce approximately the same results. The area of plant for trickling filters is much larger than for activated sludge but operation costs for the latter are quite an item considering the air required, whereas in the case of trickling filters there is relatively no cost of operation. Possibly the activated sludge method of secondary treatment will give slightly higher removals of B.O.D. but the method is more susceptible to shock than the trickling filter and may offer several other problems in control.

The high rate filters depend on higher application rates of sewage to trickling filter, and recirculation. This system makes use of the fact that the living organisms on the rock can assimilate and oxidize greater amounts of organic matter if the air is sufficient and the concentration of the organic matter not too high. Recirculation of the effluent of the high rate filter either to the filter itself or to the primary tank influent not only dilutes the concentration of the applied sewage but also brings the organic matter that has passed through the filter in contact with the organisms again. The capacity of the trickling is thus stepped up to four to five times that of the ordinary filter. Conversely it may be stated that the size of filter required is reduced to one-fourth to one-fifth of the ordinary trickling filter. Installations of these types of filters in army camps have proved to be quite effective with reductions of B.O.D. from approximately 400 p.p.m. to 40 p.p.m. Instances are reported wherein the B.O.D. is lowered to the vicinity of 10 p.p.m. The choice of particular method of application of sewage to these filters, the number of filter units in series, and the method of recirculation is one of individual preference and the difference in obtainable results is not within the scope of this general discussion. It must be borne in mind, however, that the high rate or capacity filter costs more to operate than does the standard rate filter.

Aids to Activated Sludge Treatment

Activated sludge processes may be improved by the introduction of the air at different rates to the different sections of the aeration tanks. This is known as tapered aeration and consists of adding about 50 per cent of the total air to the first third of the aeration process, thirty-five per cent of the air to the next third, and the remainder of the air to the last third of the aeration tanks. Increased capacity and increased ability to withstand shock are claimed for this system of aeration. A variation to the tapered air scheme is that of distributed input of settled sewage to the aeration units—notably the New York City practice.

The planned addition of an iron salt to the aeration process constitutes the Guggenheim process. It is generally recognized that where industry contributes moderate quantities of iron-bearing wastes to the sewer system superior results are realized and a more stable system is observed. This condition antedated the Guggenheim patents by a number of years, of course.

The Mallory, or "Oxidized Sludge Process" for secondary treatment is likewise an aeration and sedimentation process ,with the control of the relation of aeration time to sedimenattion time as the outstanding feature. The process seems to serve well under heavy loadings such as milk wastes from which B.O.D. removals as high as 99 per cent have been claimed.

The Hayes process for secondary treatment involves contact aeration in two stages. It has been successful in small installations in the Southwest but has not proved adequate in several army camp installations where the load has been high. Paddle-wheel and other mechanical aerators are mere variations of the activated sludge process. In general they have not been widely installed in this country.

As a decided aid to the activated sludge process one must look back to the development at one of the Ohio plants—that of Lima. Here the method of correcting or preventing sludge bulking by mild chlorination of the recirculated (returned) sludge was first instituted. The practice has spread and New York City, among others, makes effective use of the Lima method.

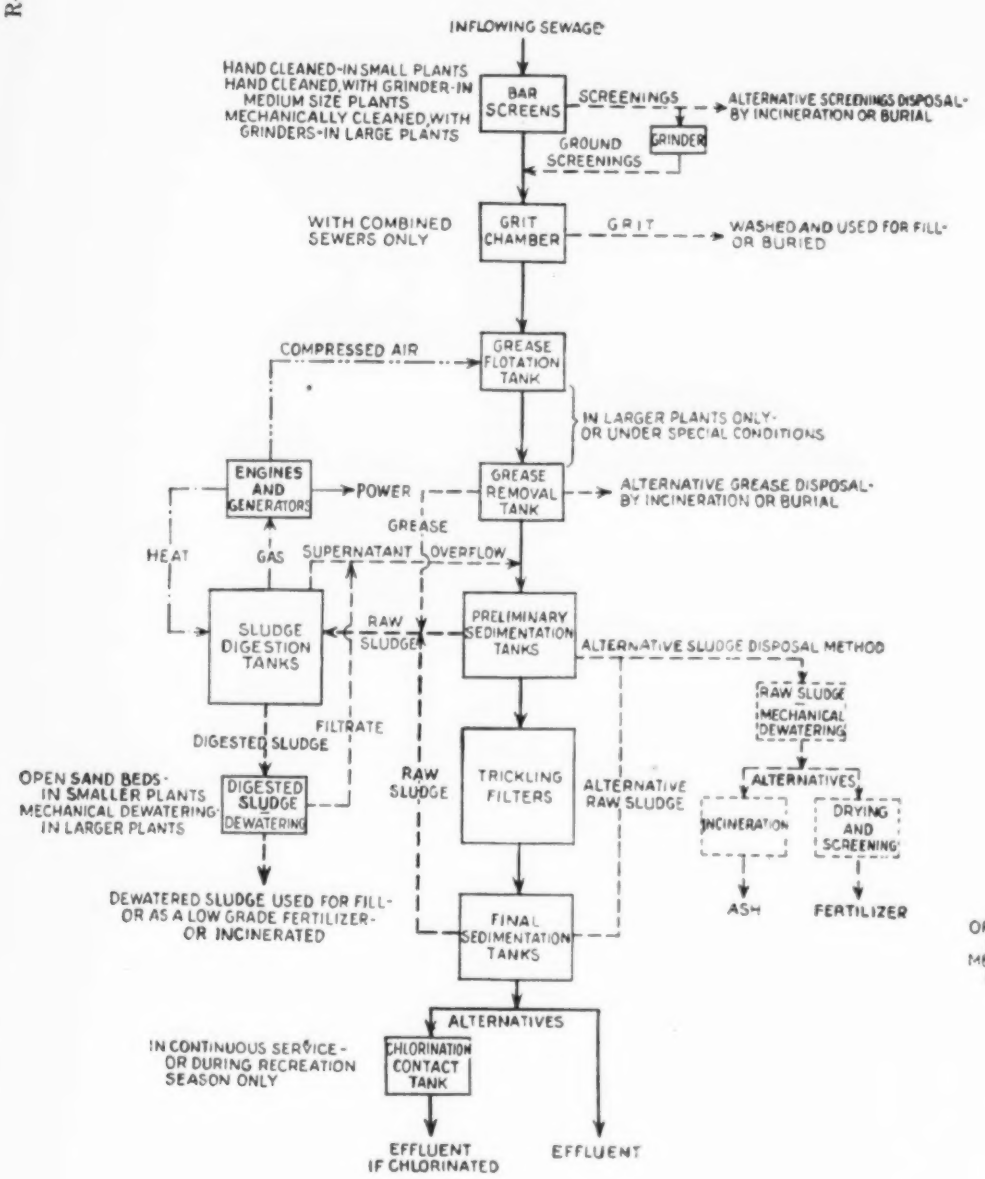
Disinfection

The extent to which disinfection is practiced is dependent principally on the use of the receiving body of water as a subsequent water supply. If the dilution is relatively small, disinfection may have to be carried to greater extent than if dilution and distance to the next water supply are great. In any event the amount of chlorine required will depend not on the number of original bacteria in the sewage or the number desired in the effluent but on the chlorine demand of the sewage. Not until the chlorine demand (amount of chlorine used up in chemical combination with substances in the sewage) is at least partially satisfied will the bacteria be killed to any extent.

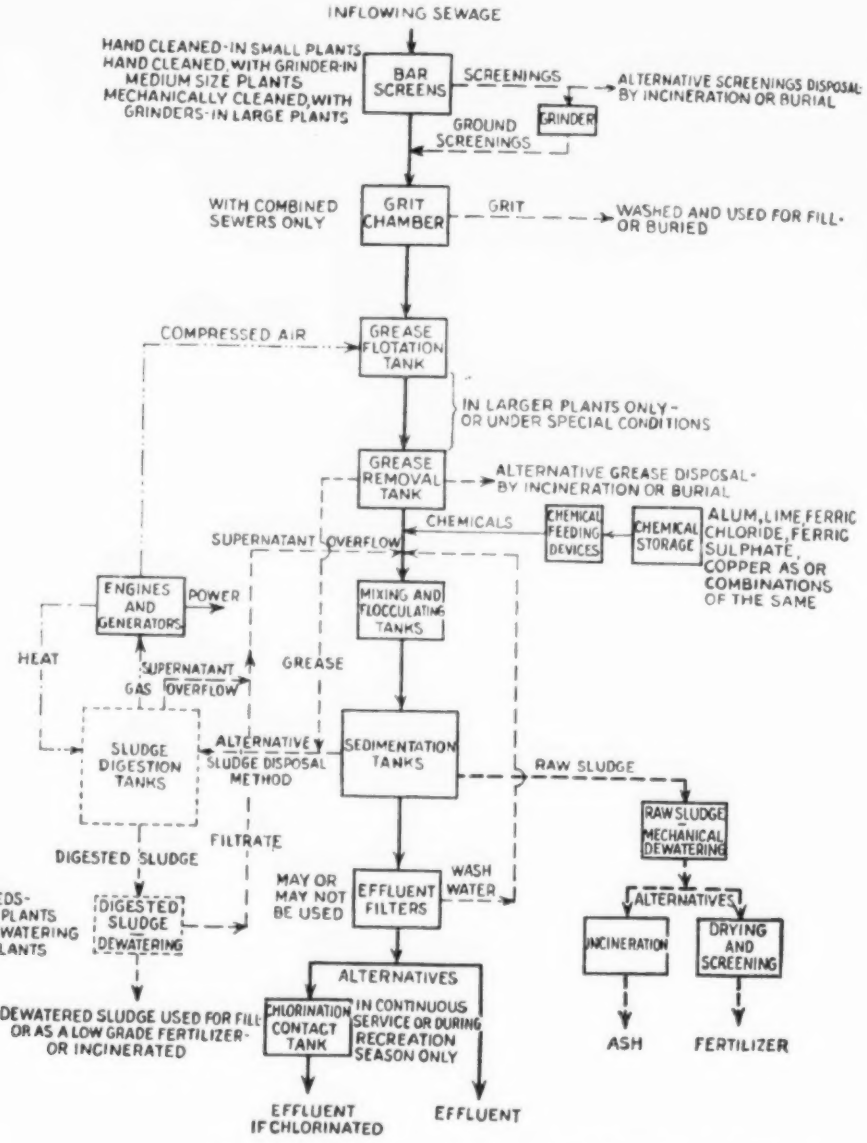
The actual kill depends on the amount of chlorine added, the amount of excess or residual produced, the type of chlorinated compounds formed in the sewage when the chlorine is added, and, most important, the time allowed for the chlorine and chlorinated compounds to act on the bacterial protoplasm. All of the bacteria are not killed, some may be revived on dilution or regrowth may occur from those left, but where the dilution is large and the normal enemies of bacteria in the receiving water sufficient, regrowth is generally not great.

In determining the efficiency of the disinfection treatment it is incorrect and misleading to report results in per cent kill based on the original number of bacteria present. This is obvious from the simple arithmetical relation that 99 per cent kill of 500,000 leaves 5,000 bacteria per unit volume whereas 99 per cent kill of 50,000 leaves only 500 bacteria in the disinfected effluent. For consistent comparisons of disinfection in winter and summer months the results should be reported in the number of unkilled bacteria after disinfection. Where the dilution is large, concentrations of coliform bacteria in the effluent may be as high as 1,000 per ml. without creating an undue load on a water filter plant below.

When dilutions are small disinfection should be carried to a greater extent. In this connection it should be remembered that



Trickling filter sewage-treatment plant, typical flow diagram



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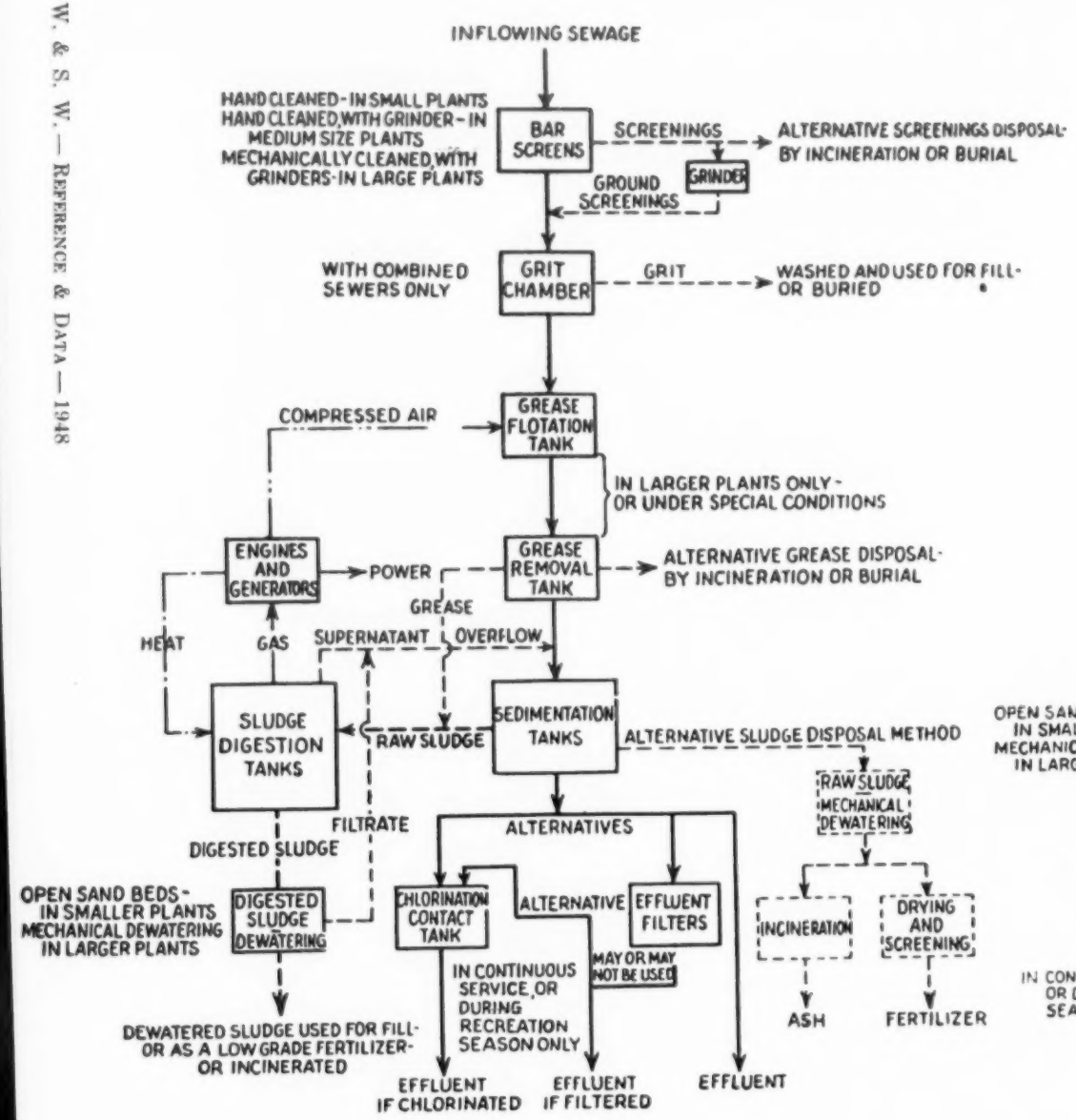
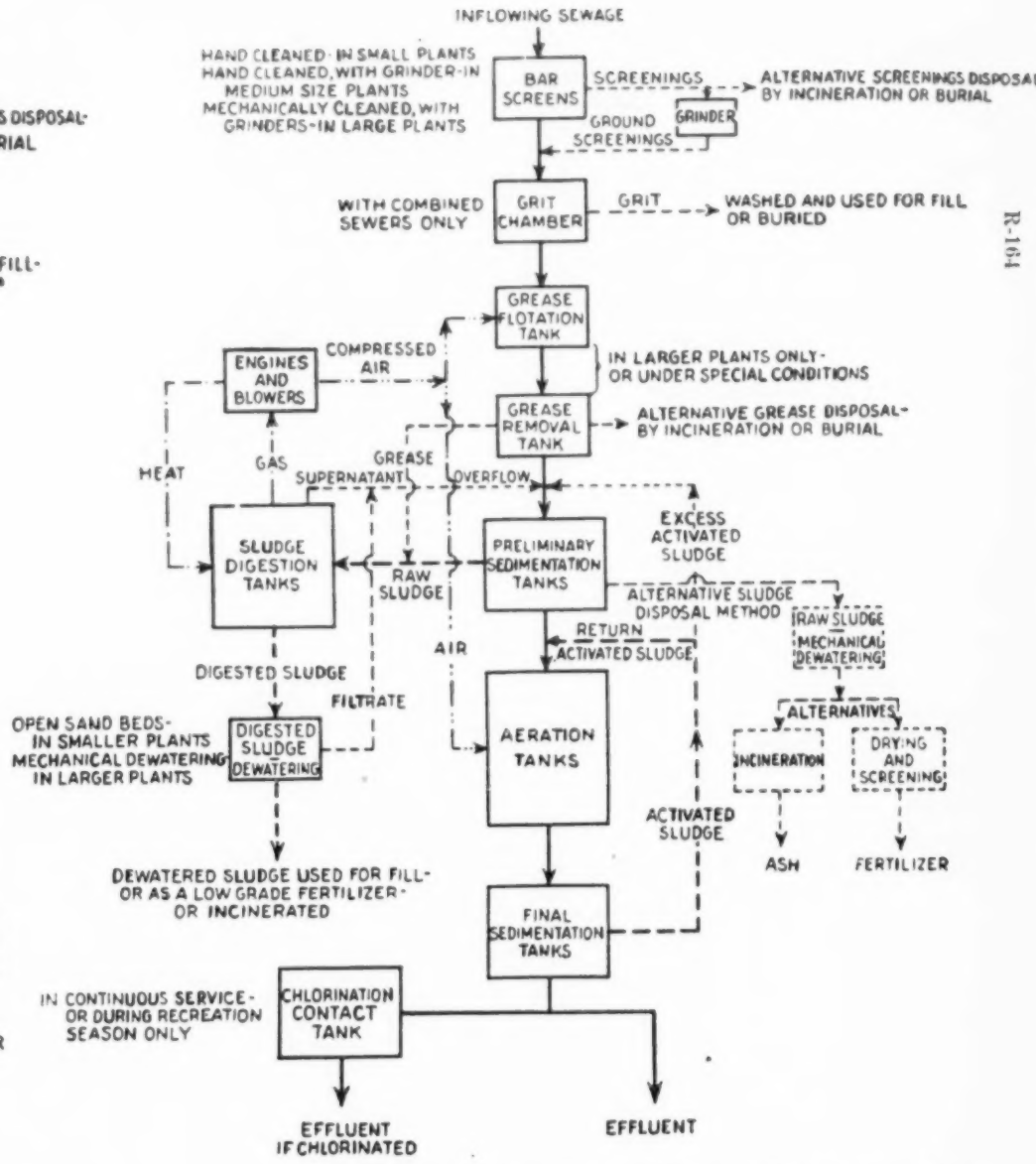
Chemical sewage-treatment plant, typical flow diagram



Plain sedimentation sewage-treatment plant, typical flow diagram



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Activated sludge sewage-treatment plant, typical flow diagram

if organic matter, suspended or dissolved, is not removed from sewage, no amount of disinfection will prevent that matter from decomposing at some later time when the bacterial population has been reestablished. Thus the discharge of chlorinated sewage into a small stream affording a low dilution merely moves the point of nuisance farther downstream or otherwise delays the bacterial degradation of the organic matter.

The point of application of the chlorine in sewage treatment may depend on several factors. If the chlorine is added up-sewer it is usually for the purpose of odor prevention, principally by killing the bacteria which will convert sulphates into hydrogen sulfide. Addition of chlorine prior to sedimentation is pre-chlorination and usually prevents odors as well as giving longer contact time for the chlorine to act as a disinfectant. It may, by preventing the formation of hydrogen sulphide in the settling tanks, also reduce the amount of chlorine used. For a normal domestic sewage it probably does not require much more chlorine for pre-chlorination than for post-chlorination.

The length of time required for disinfection (contact time) should be at least 15 minutes with a residual chlorine at the end of that time of possibly 0.5 p.p.m. If further contact time is available after 15 minutes, the residual at 15 minutes may be as low as 0.1 p.p.m., or there may even be a slight chlorine demand at that time. Chlorinated compounds formed in sewage act as bacteriacides if given sufficient time of contact. In large plants, where chlorination is important, it may be economically advantageous to utilize these chlorinated compounds rather than depend on high chlorine residuals for short contact times.

General Considerations

It is apparent from the above observations that each method of treatment has its use and application. Combinations of the processes or even of several methods in a single process may prove to be advantageous for a special problem. Usually each installation of a sewage treatment plant is a separate problem.

Disinfection is rarely practiced alone except where it is desired to eliminate a nuisance in the immediate vicinity of an outfall and allow the sewage to become more diluted before bacterial activity sets in. Primary treatment to remove suspended matter and chlorination are used when it is desired to reduce local nuisance and protect water supplies. This combination usually occurs where the dilution is great.

Primary and secondary treatment are usually combined when it is necessary to eliminate organic and bacterial nuisance completely and, as a further protection for bacterial removal, post-chlorination may be added. Secondary treatment alone is not practical. The choice of the method of carrying out these primary and secondary treatment processes may best be determined by the designing engineer when he decides on the amount of the load that each process should carry. This decision may well rest on the economics involved.

For example, at one installation a very small part of the load may be carried by a settling tank designed to remove only the coarse materials with the rest of the load to be carried by the activated sludge process. Conversely, preaeration of sewage before discharge onto trickling filters may be more economical for another plant. Choice of high rate filters as against standard rate and activated sludge likewise is a matter of economics in many instances.

Solids Disposal

The type or method used in solids disposal is quite generally decided on the basis of the economics involved. Obviously for a small community, vacuum filtration, drying and incineration are not feasible because of the smallness of the equipment required and the personnel necessary to operate such equipment. Sludge digestion and drying on sand beds remains the best method in such locations.

For these installations, without heat for digestion, the sludge may be reduced by about fifty per cent in volume and the solids 25 per cent by weight; with heat the reduction is greater. In larger cities, sludge digestion, mechanical dewatering, drying and incineration appear to offer many advantages particularly if the available land for ultimate disposal by fill is not conveniently located. Whether the sludge is sold or given away as fertilizer, depends primarily on the local market and the fertilizer value of the sludge, which, in turn, may depend on the type of treatment of sewage or the extent of digestion of the sewage solids. Barging of sludge to sea is a matter of locality as well as economics.

Inasmuch as the disposal of solids is but a by-product process of sewage treatment and not sewage treatment itself, comment on the methods of such disposal and results obtained are not primarily a part of this presentation.

Summary

In summary, it is sufficient to say that whether choosing the type of treatment process, the method of carrying it out, or the method of disposing of the important and nuisance giving solids removed, the important considerations in the decision as to choice rests on the realization and understanding of the following basic factors: The use of the receiving body of water, the capacity of the receiving body of water to receive sewage, the size of the plant site, the availability of dumping grounds for the by-product, and the economics involved in adjusting the relation between the various processes and/or the methods used between first cost and operation costs. This is but another way of saying that the choice of sewage treatment processes or methods should depend on the extent of treatment needed or desired and the best results obtainable from various combinations of these methods at the lowest possible cost for installation and for operation.

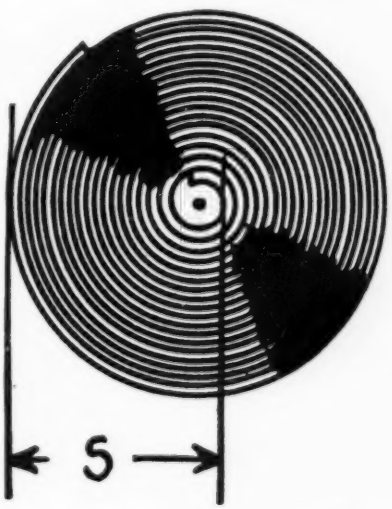
The accompanying flow diagrams show variations of combinations of methods and equipment used to effect sewage treatment.

QUICK WAY TO ESTIMATE LENGTH OF MATERIAL IN ANY ROLL OR COIL*

MOST of us are users of considerable material that comes in rolls—paper, hose, belting, cables, wire, etc.—and the question often arises, "How much material remains in that roll?" Or, "Does the original roll contain full length?"

There are numerous formulas for computing lengths in rolls, but I find that all of them are more or less complicated. We are told to find the outside diameter, the inside diameter, then subtract, multiply, and do various other things, and finally we get the answer.

Here is a simple rule of my own derivation which I contend is the simplest possible: Make the one measurement "S," as shown in the sketch herewith, in inches. Then count the number of turns. Multiply the two together and then multiply that by 0.2618. The result is the length of material in feet.



For example, if the distance "S" is 10 inches, and the number of turns is 10, the roll contains 26.18 feet.

This holds true regardless of whether the material is paper, leather, or anything else. The thickness of the material does not make any difference.

To check the rule, take an instance where we have only one single turn of belting, the distance "S" being 10 inches. The roll, then, is a true circle, and the distance "S" is the diameter of that circle. The length of the belt, of course, is equal to the diameter times 3.1416 or, 31.416 inches. Now divide that by 12 inches and you will get 2.618 feet, as per the above rule.

Even if the roll is so tightly wound that there is no visible hole through it, the rule still holds true. And, ditto, if it is loosely wound.

*Contributed by W. F. Schaphorst, M.E.

SPECIAL SEWER SERVICE CHARGES FOR INDUSTRIAL WASTES

By GEORGE E. SYMONS[1]
and
FREDERICK W. CRANE[2]

Buffalo Sewer Authority, Buffalo, N. Y.

THE cost of operation and maintenance of any sewage treatment works or disposal plant is directly affected by the quantity, character and strength of the sewage. From this basic fact stems the theory that the fairest and most just distribution of payment for the costs of sewage treatment is the sewer rental system where the user pays for services rendered proportional to the benefits derived.

The volume of water used by a service is generally the basis for sewer rentals. Industrial waste studies, however, have indicated that the concentrations of wastes discharged from certain industries into a sewer system are extremely disproportionate to the average for an entire city. Such industries, therefore, when paying sewer rental charges based on volume of wastes alone, do not adequately pay for their share of the cost of treatment of such wastes. Consequently the development of a special rate of charge for such industries based on both the volume and strength of wastes appears proper and just. A formula for such a special charge is developed and described herewith.

Buffalo, N. Y., Sewer Rental Provisions

The state of New York has authorized sewer rentals based on water consumption, fixtures, "or upon any other equitable basis." Chapter 870 of the Laws of 1939 of the State of New York covers the creation of the Buffalo Sewer Authority and repeats the above provision. Under this provision the schedule of sewer rents for Buffalo was drawn and adopted to provide that 45 per cent of the revenue was obtained from assessment against real estate and 55 per cent was to be collected on the basis of water consumption at a rate equal to 40 per cent of the cost of water. It was further provided that where water measured by a meter does not enter the sewer system the sewer rent was to be based on the quantity of water actually entering the sewer system and so determined.

In addition, the sewer rental schedule contained the following provision for special charges: "In event a lot, parcel of land, building, or premises discharges into the sewerage system sewage or other wastes which in the opinion of the Buffalo Sewer Authority contains unduly high concentrations or any substances which add to the operating costs of the sewage disposal works, then the Authority may elect to establish special rates of charge, based on the quantity of these substances, which rate of charge may be established in such a manner as the Authority may elect, or it may elect to exclude such sewage or trade wastes from its facilities."

From these provisions came two problems which confronted the Sewer Authority in regard to sewer rental charges to industrial establishments. The first of these problems arose from the protest by industries against paying sewer rental charges for water not reaching the sewer system, either due to its absorption in industrial processes, its discharge into available water-courses, or the absence of sewerage facilities. The second problem covered the supplementary charge for wastes disproportionately costly to treat.

In handling the first problem, a survey was made of all properties concerned. Where there was no sewer the matter was simple: no rental charge was made. Where water was consumed in productive processes or otherwise used up and not discharged, the attempt was made by a rational analysis to determine the average amounts so used and establish therefrom a proportion of the water consumption on which the rental charge was to be based. Little trouble was experienced with this procedure, it being generally acceptable to industry.

Each year a survey has been made of all accounts upon which the established reductions were based. These reductions have been in effect for nearly five years and no general field check has yet appeared necessary.

Somewhat as an integral part of this program has been the establishment of rental charges for water obtained from a source other than the municipal supply. Sewer rental is charged for this water at the same rate as for city water. Industries obtaining water from wells or streams are required to install and maintain meters which are read by the Sewer Authority.

Industrial Waste Survey

As an attack on the second problem, a survey of industrial establishments was made. The first step was to list all industrial concerns in the city employing more than 40 persons. There were 428 separate establishments so listed. These were divided into classes or categories according to the type of manufacturing plant or product. Twenty-seven separate classes of industries were set up. By careful consideration of these types it was possible to pick out 17 classes of industry which might discharge industrial wastes other than of sanitary character. In order of importance these were meat packers, rubber products, rendering, tanneries, breweries and malt producers, dairies, laundries, several chemical or metal industries, and others of less importance.

Each concern, 234 in all, in these groups of industries with possible industrial wastes was visited by a field engineer who obtained pertinent information concerning operations and wastes produced. These data were set forth on forms prepared for the purpose with special forms for certain industries such as the packing plants, tanneries, breweries, creameries, etc. From these data, the industries contributing the greatest waste load were determined. Samples were taken in many plants typical of a group. Other plants were sampled for a period of time, still others had been sampled in the previous years during the design and construction of the treatment works. From an analysis of the samples, the concentrations of the wastes of these industries were determined.

It was ascertained that whereas the relation of concentration to volume was disproportionate in the case of milk wastes the quantity of these wastes was too small to be of any great burden on the treatment works. Other industries, too, appeared not to warrant any special consideration as to charges, but in the case of meat packing, tanning, and rubber reclaiming it appeared that the application of special treatment charges was warranted.

Although no case is known of the application by any other city of a general formula for special sewage treatment charges of the type here considered, there are, however, numerous instances reported in the literature, where industries do pay a proportionate share of the cost of operation of the treatment works because of their excessive waste load. This instance, however, appears to be the first where a formula has been proposed for application to all industries within a municipality.

Basis for Special Charges

The Buffalo sewage treatment works is operated for the purpose of disposing of sewage and wastes in such manner as to eliminate nuisance in the Niagara River. The work which constitutes this disposal includes pumping, disinfection by chlorination, removal of solids, and the disposal of solids by digestion, dewatering, and incineration. Costs of the operation may be allocated to overhead, pumping, treatment (including sedimentation, chlorination and sludge disposal), and maintenance and repairs. The principal items of treatment cost are the cost of chlorine and the cost of chemicals and power for the disposal of solids.

Every water user paying sewer rental charges on the basis of the volume of water discharged by him into the sewers, presumably pays his proportionate share of the overhead, pumping, maintenance and repair, and normal treatment costs at the sewage treatment works. Therefore, any additional charge for service because the strength of an industrial waste is in excess of the average strength of normal sewage should be only for the purpose of reimbursing the treatment works for the extra cost entailed in the treatment of such excess waste load. The extra cost can be definitely assigned to excess chlorine consumption and to excess chemicals and power used for the disposal of solids. The quantities of these materials required in excess of the normal sewage requirements can be ascertained for an individual waste by analyzing the waste to determine its chlorine demand and its suspended solids content.

Utilizing the above established principle that additional special charges should be based only on the extra cost of treatment of excess waste loads, the following considerations entered into the development of a general formula for the determination of such

[1]Consultant, Buffalo Sewer Authority—Now Associate Editor, *Water and Sewage Works.*
[2]Gen. Manager, Buffalo Sewer Authority.

charges applicable to all industries. As indicated this formula should be based on the chlorine demand and suspended solids content of the particular waste, and on the cost of chlorine and on the cost of chemicals and power for the disposal of solids. The formula will, therefore, be such as to take cognizance of any change in the character of the waste discharged by any industry as well as of any change in the cost of operation of the treatment works. Consequently, periodic sampling and analyses of all wastes subject to special charges may be necessary to establish their average chlorine demand and suspended solids content. Finally, the formula should be such as to develop a rate of special charge which will be applicable to the volume of the wastes.

Derivation of General Formula

The approach to the derivation of the formula is given in the following steps:

1. Develop the formula to calculate special charge rate (R) in cents per volume unit of 1,000 cu. ft. of waste.

2. Assert the principle that water user is entitled, through payment of ordinary sewer rent, to concentrations of chlorine demand and suspended solids as normally present in sewage received at the treatment works, but that water user must pay for chlorine demand and suspended solids in his wastes above such normal concentrations. Therefore, introduce into the formula differences between concentrations of chlorine demand and suspended solids in the waste and concentrations of like substances normally present in sewage.

3. The difference between chlorine demand of waste and chlorine demand of normal sewage, reduced to pounds per 1,000 cu. ft. of waste by multiplying the concentration difference by a factor (F), is the excess chlorine demand per volume unit of waste for which the special charge should be made.

4. The difference between the amount of suspended solids in waste and suspended solids in normal sewage, reduced to pounds per 1,000 cu. ft. of waste by multiplying the concentration difference by factor (F), is the excess suspended solids per volume unit of waste for which the special charge should be made.

5. To obtain special charge rate (R), the pounds of excess chlorine demand per volume unit of waste is to be multiplied by the cost of chlorine per pound (Pc), and added to the pounds of suspended solids per volume of waste multiplied by the cost of disposing solids (Ps), i.e., cost of chemicals and power for chemical conditioning and incineration, exclusive of overhead.

6. The general formula applicable to all concerns and to all industries thus becomes:

$$R = FPc\ (C - Nc) + FPs\ (S - Ns)$$

Where R = rate of special charge in cents per 1,000 cu. ft. of volume of waste.

F = factor for converting parts per million to lbs. per 1,000 cu. ft.

Pc = contract price of chlorine in cents per lb.

C = concentration, in parts per million, of chlorine demand of the waste.

Nc = normal dry weather chlorine demand of raw sewage in parts per million of sewage as received at the treatment works.

Ps = cost in cents for chemicals used in sludge conditioning and for power for disposal of solids resulting from one lb. of suspended solids received in the raw sewage at the sewage treatment works.

S = concentration, in parts per million, of suspended solids of the waste.

Ns = normal grit-free suspended solids in parts per million of raw sewage as received at the treatment works.

The factor Pc may vary from year to year according to the contract price for chlorine and the factor Ps may vary according to the costs of chemicals and power involved in the disposal of solids as determined from operations for the previous year or previous two years. The factors C and S may be established by analyses (periodic or otherwise) to obtain average concentrations to be applied for any agreed length of period.

Determination of Values of Factors in Formula

1. Inasmuch as 1,000 cu. ft. equals 62,400 lb., one ppm. equals 0.0624 lb. per 1,000 cu. ft. Therefore, F = 0.0624.

2. The 1942-43 contract price of chlorine is $43.60 per ton or 2.18 cents per pound. Therefore, Pc = 2.18.

3. Concentration of chlorine demand (C) in ppm. in industrial waste is to be determined by standard methods of analysis.[1]

4. Chlorine demand of raw sewage or normal dry weather flow averages 5.6 ppm.[2] Therefore, Nc = 5.6.

5. Cost of disposing of solids is based on incoming grit-free suspended solids. Grit-free sewage contains none of the grit from streets washed by storms and represents approximately normal suspended sewage and waste solids as received at the treatment works. Treatment works studies at Buffalo indicate approximately 25 per cent of the total power consumption is for incineration operations. Buffalo data for years 1939-40 and 1940-41 were averaged to give representative basis for cost determination as follows:

Year	Sewage Flow M.G.D.	Grit-Free p.p.m.	Susp. Solids Lb./Yr.	Cost of Chemicals for Sludge Conditioning Per Year	Cost of 25% of Power Used Per Year
1939-40	135	157	.....	$21,160.66	$11,712.68
1940-41	140	158	.....	18,382.43	13,069.89
Average	137.5	157.5	65,923,700	19,771.54	12,391.28

$$\text{Therefore, } Ps = \frac{\$19{,}771.54 + 12{,}391.28}{65{,}923{,}700} = \$0.000488 = .0488 \text{ cents}$$

6. Concentration of suspended solids (S) in ppm. in industrial waste is to be determined by standard methods of analysis.[1]

7. Suspended solids in grit-free sewage average 157 ppm. Therefore, Ns = 157.

When these factors for present costs and sewage concentrations are applied the general formula takes the following form:

$$R = 0.136\ (C - 5.6) + 0.00305\ (S - 157)$$

Thus it is seen that, in the application of the formula under present Buffalo conditions, the special charge rate per 1,000 cu. ft. of wastes would be 0.136 cents for each part per million of chlorine demand and 0.00305 cents for each part per million of suspended solids above the normal concentration of each in the raw sewage as received at the treatment works. It is to be especially noted that the industrial waste load is included in the normal concentration values of 5.6 ppm. for chlorine demand and 157 ppm. for suspended solids content of the sewage as received at the treatment works.

There may be special cases to consider in the application of the formula. Some industrial wastes may change characteristics when diluted with a relatively large volume of sewage. In such cases the total daily chlorine demand and suspended solids loads may decrease, or increase. The change may be shown by experimental tests on mixtures of the waste and sewage prepared in the ratio of their respective volumes.

Where experimental evidence shows the chlorine demand and suspended solids load to decrease or increase on dilution, the equivalent value or effective concentration of this decreased or increased load in ppm. concentration may be calculated for the original waste. On negotiation with the company concerned it may be agreed that the equivalent concentration of the waste is to be used in the formula for determining the special sewer rental rate. In cases of this kind, frequent analyses and experiment should be made to establish the validity of the concentration used.

Suggested Limits for Application of Special Charges

It does not appear economically feasible to apply a special charge when the rate is less than 1 cent per 1,000 cu. ft. of volume of waste. From considerations of the amount of work involved, it would also appear evident that it is not economically worth while to make special charges in any case where the total reimbursement does not cover by a considerable margin the cost of investigations and necessary bookkeeping.

On the basis of Buffalo's current costs, if suspended solids were not present in a waste, the concentration of chlorine demand would have to be 13 parts per million (2.3 times the present normal sewage value of 5.6 parts per million) to produce a special charge rate or R value of 1 cent per 1,000 cu. ft. Likewise, if chlorine demand were absent, the suspended solids content of a waste would have to be 485 p.p.m. (approximately three times the present normal sewage value of 157 p.p.m.) to produce a special charge rate or R value of 1 cent per 1,000 cu. ft. It is obvious that smaller concentrations of these items when in combination with each other will produce a value for R of 1 cent or more.

The Grease Problem

Despite the fact that the Plumbing Code of the City of Buffalo expressly forbids and prohibits the discharge, without prior treatment, of any grease, fats, oils, etc., into the sewer system, accumulations of such materials are found in the sewers and to appear at the treatment plant.

The presence of grease, fats and oil in sewage reaching the treatment plant is objectionable since these materials form trou-

blesome scum in channels and on sedimentation tanks interfere with biological processes, and cause problems of excessive heat on incineration. Although they add to the complication and cost of handling and disposing of solids, nevertheless it does not appear possible to determine the direct added treatment cost due to their presence in the sewage, except as they are included, at least partially, in the suspended solids.

The tendency of grease, fats and oil to clog the sewers is also a highly objectionable characteristic of these materials. Wherever the source of these materials in sewers is ascertainable, it is possible to determine the cost of cleaning the sewers whose clogged condition is directly attributable to these wastes. However, it is quite evident that some accumulation of these materials likewise occurs throughout the length of the dry-weather sewer system and causes troublesome conditions at weirs, siphons, etc. No direct cost can be assigned to remedying these latter conditions.

Summary

From the standpoint of feasibility of application and of effecting an equitable distribution of costs of treating excess waste loads, it has appeared possible to develop and apply a general formula for special rates of charge to any industrial plant whose waste load has a concentration measured in chlorine demand or suspended solids content of approximately three times that in the sewage as now received at the Buffalo treatment plant. This general formula is such as to permit adjustments of the special rates at periodic intervals accordingly as variations in the concentration of the waste received from an industrial concern or as variations may occur in the costs of treating such wastes.

It is proposed that the application of special rates or charge as developed from the general formula be made in the same manner as normal sewer rental charges, except that the special charges be separately billed. Under this procedure, periodic sampling and analysis of the wastes of the individual concerns may be necessary to maintain the rates of charge on an equitable basis. This will serve both as a protection to the interests of the Authority and to avoid penalizing an industrial plant which improves the character of its wastes. The choice of periodicity of sampling and the extent of sampling may be left to negotiation.

There appears to be no direct way in which a special rate of charge can be determined to cover the costs caused by the discharge of grease into the sewers. It has been recommended that provision be made to charge any concern for the cost of cleaning a sewer or sewers when the clogged condition of the same is directly attributable to the wastes discharged into the sewers by such concern. This provision should be broad enough to cover other materials which also tend to obstruct the sewers.

Whereas the formula herein described has been developed for application to the specific case of Buffalo, N. Y., where disposal consists of primary sewage treatment and disinfection with incineration of solids, there seems to be no reason why the same general type of formula could not be developed in any municipality for any type of sewage treatment and problem involved.

REFERENCES

(1) Standard Methods for the Examination of Water & Sewage. 9th Ed.

(2) G. E. Symons, R. W. Simpson, & S. R. Kin. "Variations in the Chlorine Demand of Buffalo Sewage," Sewage Works Journal 13, 249-64 (1941).

SIZE AND SLOPE OF SEWERS

By Simplified Formulae Approximating Kutter's

By **BENJAMIN EISNER**, *Chief Engineer*

Clay Sewer Pipe Association, Pittsburgh, Penna.

KUTTER'S formula for computation of flow velocities in open channels is preferably solved by graphs to avoid the lengthy arithmetic necessary to compute "C." Because pipe sewer problems are but a small portion of the wide field of application of the Kutter formula, and since extreme precision is not warranted in sewer and drain design, it is possible to approximate the Kutter formula for such problems by matching a simple curve to the complex one expressed by the formula. Such matching is possible only because short lengths of the curves are used. For sewers and drains "n" is usually taken at either 0.013 or 0.015: "v" is required to exceed 2 feet per second, pipe diameters generally fall between 8 and 36 inches, and slope rarely reaches 10 per cent grade.

If within these limitations, values for "s" are plotted against "d" for any given velocity, the resultant curve closely approximates a hyperbola. If too, for later convenience, "s" is measured in feet per thousand, and "d" in inches of diameter, the hyperbolic curves may be expressed in the following manner:

For "n" = 0.013: $v = 1/2\sqrt{s}\ \sqrt{d-4.5}$ (A)

For "n" = 0.015: $v = 4/9\sqrt{s}\ \sqrt{d-5}$ (B)

Of course, both cases apply only to pipes running full or half-full. Accuracy of results by use of these approximations is indicated in tables I and II which cover the range of application outlined above.

Use of these approximations with a slide rule is believed to be quite as rapid as use of charts for the solution of Kutter's formula. Although there is a slight loss of accuracy, the error thus introduced is less in amount than the errors accompanying estimates of future flow and choice of coefficient "n."

So far, no mention has been made of the term Q as used in $Q = Av$. If Q is measured in million gallons daily, if "d" is used as before, to denote pipe diameter in inches, and if the pipes are considered to run full, $Q = Av$ can be reduced to

$$d = 17\sqrt{\frac{Q}{v}} \qquad (C)$$

which is easily converted to a form suitable for rapid determination of pipe size for minimum self-cleaning velocity. If velocity of two feet per second is the lower limit, this expression reduces to $d = 12\sqrt{Q}$, for full flow, which can be further simplified by expression as "diameter in feet for 2 feet per second velocity equals the square root of the full flow in millions of gallons daily." Similarly the previous approximations for "s," at 2 feet per second velocity reduce to the fllowing—

For: $(n = 0.013)\quad s = \dfrac{16}{d-4.5}$ For: $(n = 0.015)\quad s = \dfrac{20}{d-5}$

Thus for minimum self-cleaning velocities determination of diameter and slope is rapid.

Commitment to memory of approximatoins (A) and (C) permits rapid sewer and drain design when flow charts are not readily available. For the slight loss of accuracy, speed and convenience are gained. In practical use "d" is assumed and approximate "v" determined as in (A).

This value is substituted in (C) and the result "d" is compared with the assumption. With slight practice the second trial value of "d" will be found to satisfy both equations within practical limits.

Comparison of Velocities of Flow in Conduits (Running Full or Half Full) Computed by Approximate Formula and Kutter's Formula

TABLE I (n=0.013)

SLOPE (Feet per 1,000 ft.)	DIAMETER (Inches)				
	8	12	18	27	36
0.1			**0.9** 1.0	**1.3** 1.3	**1.6** 1.6
1	**0.9** 0.9	**1.4** 1.4	**1.9** 1.9	**2.6** 2.5	**3.3** 3.0
5	**1.6** 1.7	**2.4** 2.4	**3.2** 3.2	**4.1** 4.3	**4.9** 5.2
10	**3.0** 3.2	**4.3** 4.3	**5.5** 5.6	**7.5** 7.8	**8.9** 9.6
50	**9.1** 9.5	**7.5** 7.5	**10.1** 10.1	**13.9** 13.5	
100	**9.4** 10.0	**13.7** 13.8			

TABLE II (n=0.015)

SLOPE (Feet per 1,000 ft.)	DIAMETER (Inches)				
	8	12	18	27	36
0.1			**0.9** 0.9	**1.3** 1.3	**1.4** 1.6
1	**0.8** 0.6	**1.2** 1.1	**1.6** 1.5	**2.3** 2.1	**2.9** 2.3
5	**1.3** 1.4	**1.9** 1.9	**2.8** 2.7	**3.6** 3.6	**4.3** 4.4
10	**2.4** 2.6	**3.4** 3.6	**5.0** 5.0	**6.6** 6.6	**7.8** 8.1
50	**6.2** 6.1	**5.9** 6.2	**8.8** 8.5	**11.6** 11.2	**13.6** 13.8
100	**7.7** 8.2	**11.7** 11.5	**16.9** 15.5		

Bold face figures indicate approximation. Light face figures are results from chart of Kutter's Formula.

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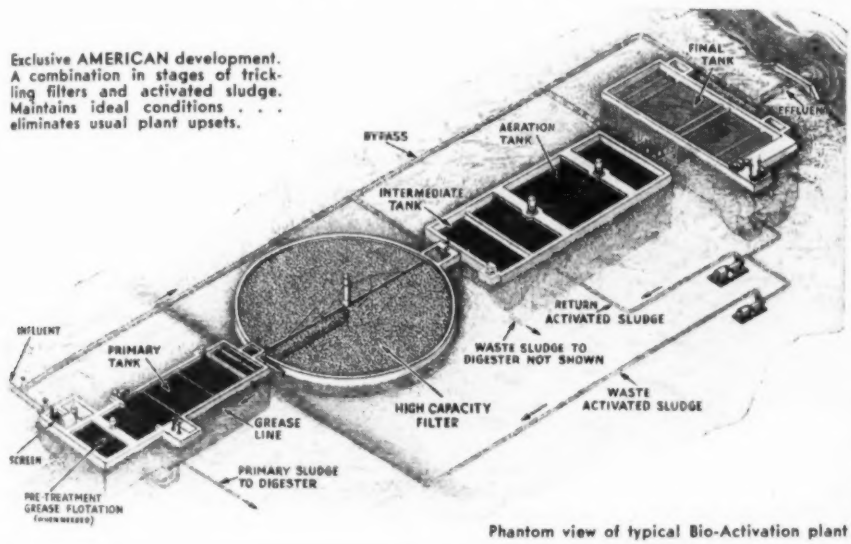
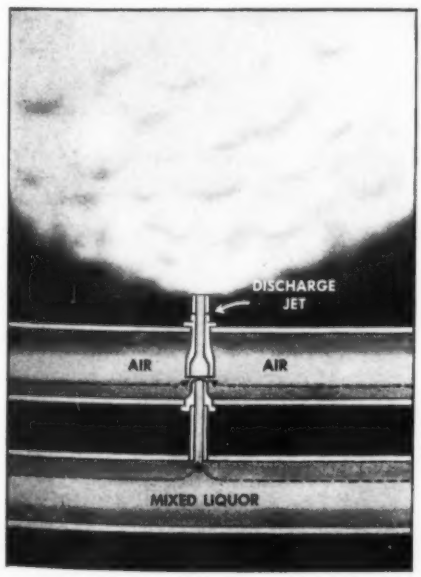


Phantom view of typical Bio-Activation plant



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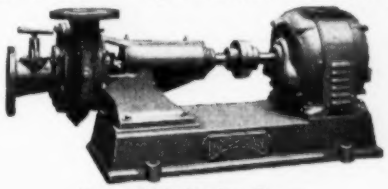

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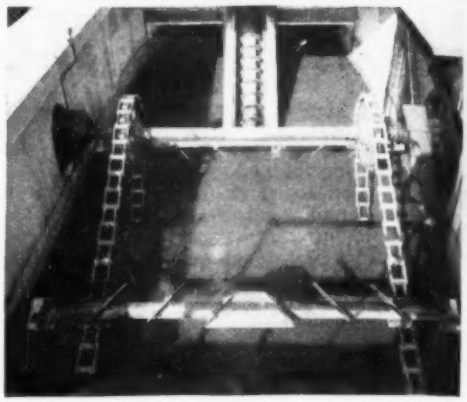

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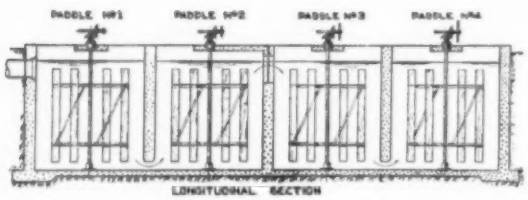

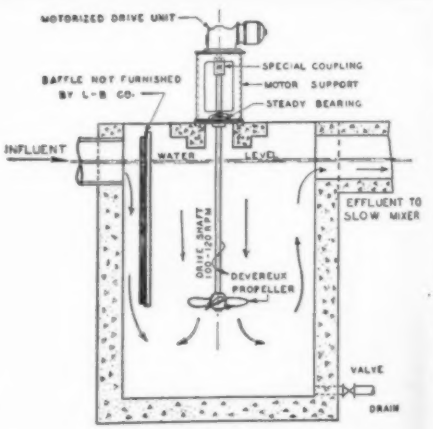

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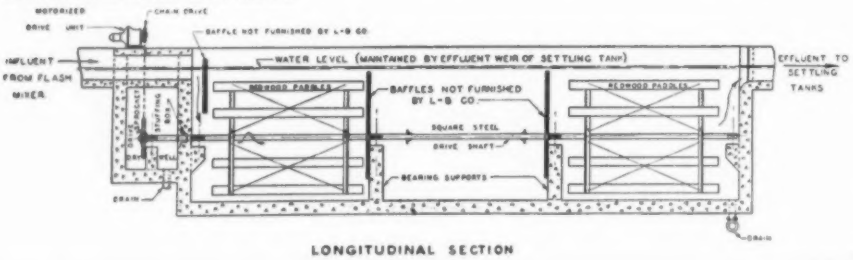

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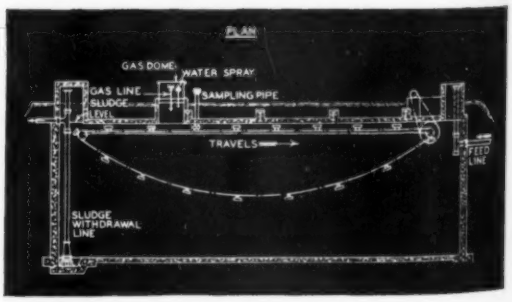

Sewage and Industrial Waste Problems

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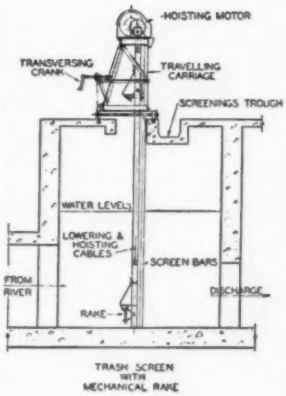

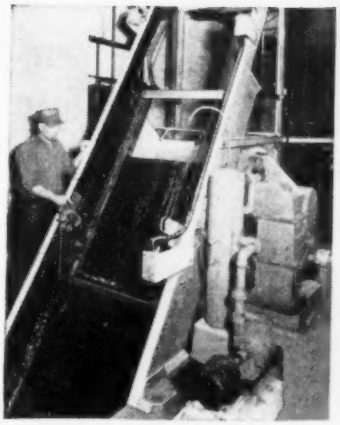

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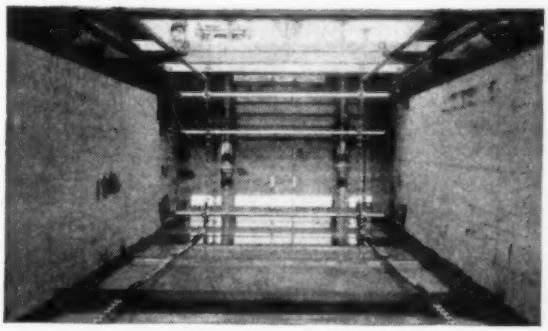

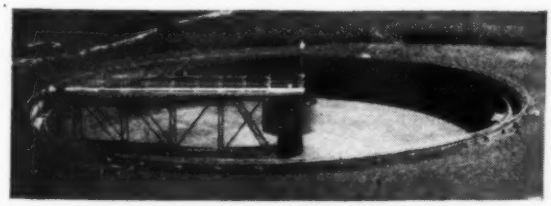

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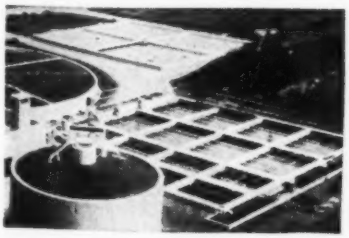

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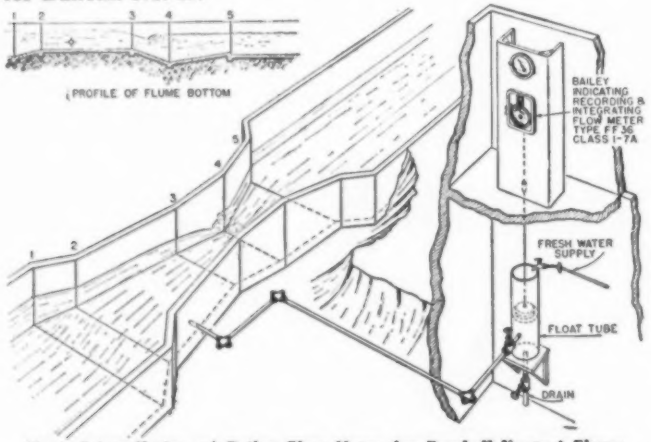
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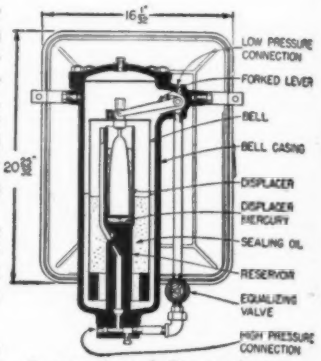
Described in bulletin No. 301-B.



Section through flow mechanism of Air Meter

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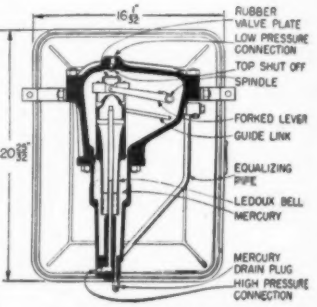

Section through flow mechanism of Ledoux Bell type Register.

Remote Registers — Bailey Telemetering Systems make it possible to operate remote indicating, recording or integrating registers, which may be installed at a distance from the direct connected unit. These registers may measure flow in individual lines, total flow, liquid level, pressure and other factors.

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Bailey control is fully automatic, dependable and exceptionally flexible.

Bailey Recorder—Controller.

Bailey Registers measuring Sewage, Sludge and Air in an operating gallery at Easterly Sewage Treatment Works, Cleveland, Ohio.

MEASUREMENT OF FLOW IN OPEN CHANNELS

Parshall Flume, Using "Free Flow" Condition Gives Ready Measurement for Silt-Bearing Waters or Sewage

By J. TARRANT
Ruislip, England

THE Parshall measuring flume described by R. L. Parshall in Bulletin 423 of the Bureau of Agricultural Engineering has been proved to be a reliable method of measuring the flow of water in channels, and is especially useful where the water carries solids in suspension.

The older method of measuring the flow in channels by means of weirs or notches has the disadvantage that the barrier across the channel carrying the weir or notch forms a pocket where sediment may collect, with detrimental effects on the accuracy of measurement. The Parshall flume has no such obstructions and can be so proportioned that the velocity of flow carries forward the silt and maintains a clear channel.

There is also the question of head loss which, when pumps are used for delivering water or sewage on to a plant or when it is desired to avoid pumping, may be an important consideration. The head lost in a flume, by utilizing the full allowable submergence at the outlet, is very considerably less than that imposed by the use of a weir or a notch.

The principle of measurement is similar to that of the weir in that the head over a sill is measured, and this measurement is used either alone or in conjunction with a second head reading taken downstream of the sill for the estimation of the rate of flow. The use of two readings means that (because the downstream head may rise to a value equal to about 95 per cent of that upstream) the loss of head can be reduced to an extremely low figure, but the indicating or recording of the flow is complicated as compared with either by a single head reading.

Head Loss

When measuring the flow by a single head (the "free flow" condition) the loss of head may not be reduced below a figure which is a percentage of the head on the sill for each rate of flow. (The figures are given later.) This head loss is such that the level of water downstream is not allowed to rise to the point where it will have any effect on the volume of the discharge as indicated by the head on the sill. That is to say, for a given head on the sill the flow remains constant as the tail or downstream water level is progressively raised until, at a certain submergence, the flow begins to decrease.

So, within the limit imposed by this condition of noninterference, the discharge may be measured by a single head reading. The maximum level to which the downstream water may be allowed to rise is 50 per cent of the upstream head in the case of 3-in., 6-in. and 9-in. flumes, and 70 per cent for flumes from 1 ft. to 8 ft. in width. The loss of head is the difference in level between the water entering and leaving the flume, and this is equal to 50 per cent or 30 per cent of the head on the sill, according to the width of the throat.

In earlier issues of *Water Works and Sewerage* the use of the Parshall flume for the measurement of sewage flows has been described and the formulas given. The selection of the most suitable of flume for any given set of flow conditions, involving consideration of channel width, depth of flow in the channel, loss of head permissible, possible future increases in the flow, and so on, is assisted by the use of charts, the final sizing of the flume being done by calculation from the formula.

The accompanying chart, Fig. 3, has been prepared for sizing flumes of 3-in., 6-in., and 9-in. wide and from 1 ft. to 8 ft. wide (the width referred to is always the throat width). The tables in Bulletin 423 give the flows for various heads for flumes measured in exact feet of throat width from 1 ft. to 8 ft., but for the selection of a flume to suit particular conditions of head and discharge it may be useful to be able to select a size which may be anywhere between 1 ft. and 8 ft. wide.

In selecting a flume for a given duty and set of conditions it will be found that the smaller W is made for a certain discharge the greater will be the head loss, assuming that allowance for the maximum submergence is made in each case. The depth of flow in the channel downstream of the flume and the level of the floor in the chanel may be convenient points to take as the basis for sizing, working back to the water level upstream and the level of the crest or sill. Use of the chart means that a number of different sets of conditions may be tried out quickly to determine the best size of flume for the job in hand.

The range of heads covered by the chart is the same as shown in the various tables in Bulletin 423 and the formulas are those there given for "free flow" conditions, viz.:

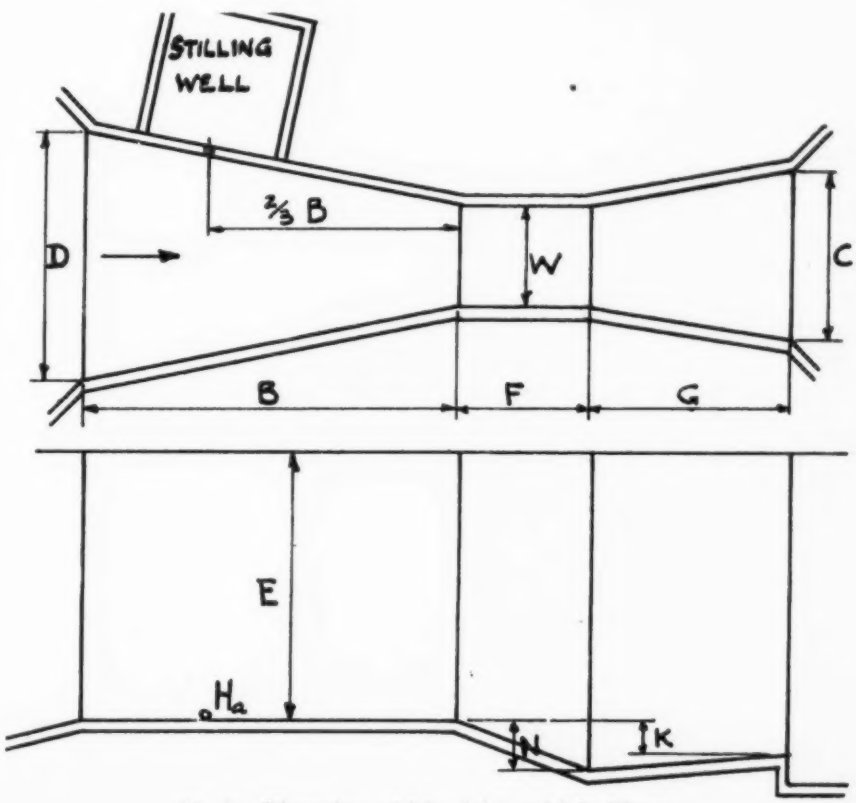

Fig. 1.—Dimensions of 3-in., 6-in., and 9-in. Flumes

TABLE I

Dimensions in 3-in., 6-in., and 9-in. Flumes (See Fig. 1)

W	B	C	D	E	F	G	K	N
in.	ft.-in.	ft.-in.	ft.-in.	ft.-in.	ft.-in.	ft.-in.	in.	in.
3	1-6	7	10-3/16	1-3	6	1-0	1	2¼
6	2-0	1-3½	1-3½	1-6	1-0	2-0	3	4½
9	2-10	1-3	1-10⅝	2-0	1-0	1-6	3	4½

TABLE II

W	1.0	1.25	1.5	1.75	2.0	2.25	2.5	2.75
n	1.522	1.5308	1.5381	1.5443	1.5496	1.5544	1.5586	1.5625
W	3.0	3.25	3.5	3.75	4.0	4.25	4.5	4.75
n	1.5660	1.5693	1.5724	1.5752	1.5778	1.5803	1.5826	1.5849
W	5.0	5.25	5.5	5.75	6.0	6.25	6.5	6.75
n	1.5870	1.5890	1.5909	1.5928	1.5945	1.5962	1.5978	1.5994
W	7.0	7.25	7.5	7.75	8.0			
n	1.6009	1.6204	1.6038	1.6052	1.6065			

Table II gives the value of "n" $= 1.522^{.026}$ for widths of flume from 1 foot to 8 feet in steps of 3 inches.

"W" is given in feet, and "n" is the value of the index to "H" in the formula, $Q = 4WH^{1.522W^{0.026}}$ which is thus reduced to, $Q = 4WH^n$, using second-feet units, so facilitating logarithmic calculation.

For 3 in. flumes—$Q = 0.992Ha^{1.547}$
For 6 in. flumes—$Q = 2.06\ Ha^{1.66}$
For 9 in. flumes—$Q = 3.07\ Ha^{1.55}$
For 1 ft.
to 8 ft. flumes—$Q = 4\ W\ Ha\ 1.1522\ W^{0.026}$

where

Q is the dicharge in cubic feet per second.
W is the width of the throat in feet.
H is the head on the sill in feet.

These formulas, of course, are the same as given in Bulletin 423 but are repeated here in order to make these notes as complete as possible.

It is mentioned in the bulletin that the width of the throat will, generally, lie between one-third and one-half the width of the channel, and this makes a good starting point, but this rule will not apply for extremely wide and shallow, or deep and narrow, channel flows, and will be governed by considerations of head loss, and so on, as mentioned earlier. The Bulletin recommends in considering the loss of head that a submergence of 60 per cent be taken as the practical limit for flumes between 1 ft. and 8 ft. wide so as to allow for possible fluctuations in flow before the conditions affecting the accuracy of measurement by the single head reading would be reached. Similarly the practical limit for 3 in., 6 in. and 9 in. wide flumes might be taken as 40 per cent or 45 per cent submergence.

The head Ha over the sill or crest is read at a point two-thirds the distance back from the crest to the inlet of the converging section, and, if a connection is taken to a stilling well for the float operation of an indicator or recorder, the connection is taken through either side wall, just above the floor, the end of the pipe being flush with the wall face.

Table I and Fig. 1 give the dimensions of the 3-in., 6-in. and 9-in. flumes, those dimensions common to all three sizes being given in the figure.

Dimensions of flumes between 1 ft. and 8 ft. wide are given for exact feet widths in the Bulletin, but for calculating the dimensions "B" and "D" for flumes of any width between these limits the formulas which follow are offered. The throat width "W" is used as the basis of each formula and so when the width has been decided the other dimensions are quickly obtained. "W" is taken in inches and the other dimensions are derived in inches. The results compare very closely with the dimensions given in the Bulletin for widths of even feet, and so may be used for intermediate widths.

B = 0.49 W + 47"
C = W + 12" (See Table I. Bull. 423)
D = 1.196 W + 18.84"

Thus, for a 30-in. flume: B = 0.49 × 30 in. + 47 in. = 61.7 in.; C = 30 + 12 = 42 in., and D = 1.196 × 30 + 18.84 in. = 54.72 in. Position of dimensions "W," "B," "C," and "D," and dimensions which are common to all sizes between 1 ft. and 8 ft. flumes are given in Fig. 2.

Using the Chart: Fig. 3:

The use of the chart is illustrated by the lines A.B.C. and D.E.F. which cross the scales. A.B.C. shows that a flume 24 in. wide will pass 8.0 cu. ft. per second or 5.17 mgd. (U. S.) when Ha is 12 in. Line D.E.F. is used to show the flow through a 9-in. flume when the head is 6 in. This equals 1.06 cu. ft. per second or 0.685 mgd.

Similarly, for 3 and 6-in. flumes, the flow is obtained by using the appropriate points for the width and the corresponding scale for the head. Thus, it will be clear that one scale is used for the flows for all sizes with separate scales for the head for 1 ft. to 8 ft., 3, 6 and 9-in. widths, respectively. A scale of widths is given for 1 ft. to 8 ft. with points for 3, 6 and 9-in. widths, no intermediates being given for these, each of which is set out according to the formula given for that size.

In the stilling well, shown in Figs. 1 and 2, a scale may be placed against the wall, having its zero at the level of the flume crest. This may be calibrated from the chart or from the calculated rate from the formula, depending on the degree of accuracy required. A mechanical indicator operated by a float in the well may be mounted above the well to give a continuous indication or record of the flow.

The location of the measuring flume in the channel is a matter requiring some consideration. The Bulletin suggests that a certain length of straight channel should precede the flume but is not specific about this. The approach should be arranged so as to give conditions free from turbulence.

The velocity of the water in the channel leading to the flume should be less than the critical velocity, and, with this limitation, a variation of as much as 300 per cent in the velocity of approach was found to have no effect on the discharge. Where the depth in the approach channel is low and the velocity high, the selection of a flume size can be made which will increase the head, and therefore the depth in the approach channel, so decreasing the approach velocity to a suitable figure.

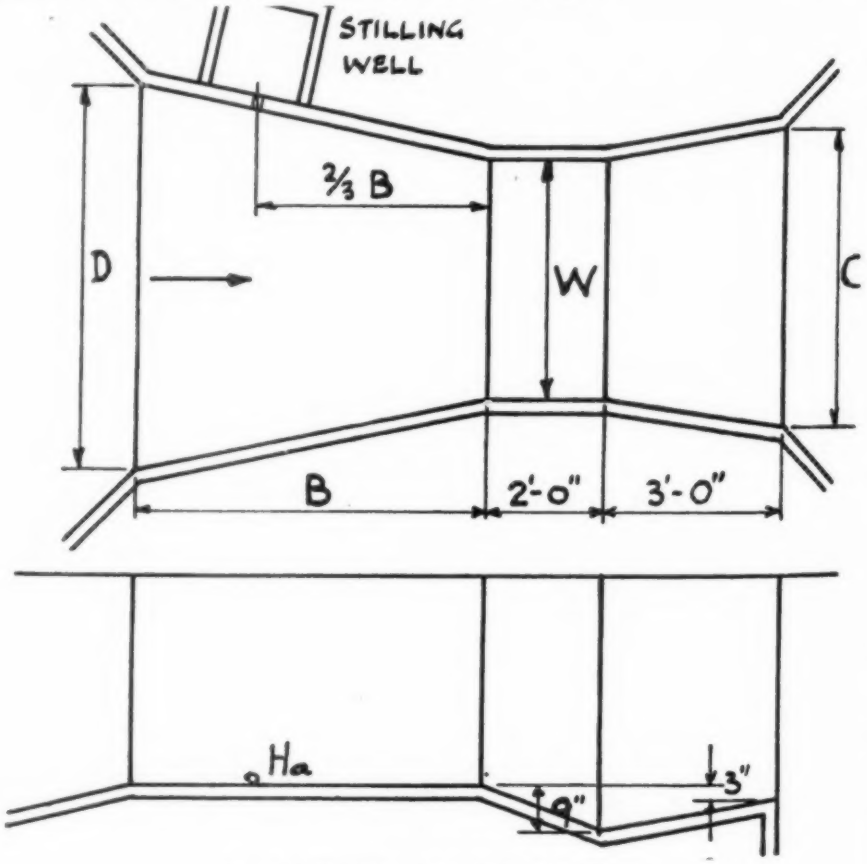

Fig. 2.—Dimension of 1 ft. to 8 ft. Flumes (See Formulas for B, C, and D)

Larger Flumes—Dimensions of wider flumes, that is, of widths greater than 8 ft., are not calculated from the formula as given above. Figures for flumes from 10 to 50 ft. in width are given in Bulletin 386 of the Bureau, the largest having a capacity up to 3,000 cu. ft. per second.

References:

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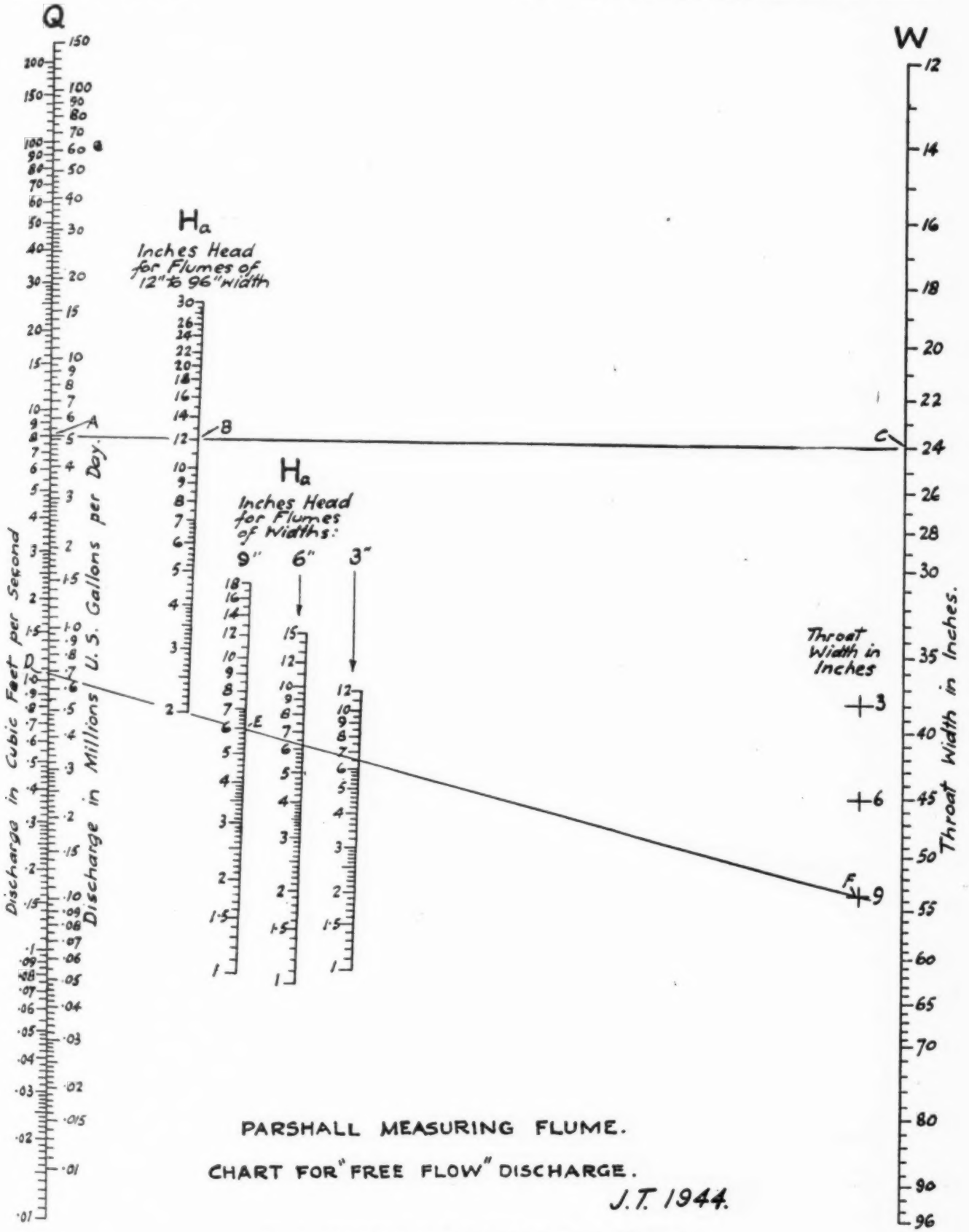

Fig. 3.—Chart for Free Flow Discharge of Parshall Flume

DESIGN AND OPERATION OF GREASE INTERCEPTORS*

By F. M. DAWSON and A. A. KALINSKE
Iowa Institute of Hydraulic Research

GREASE interceptors (or "grease traps" as they are sometimes called) have been used in plumbing drainage systems for many years. They are frequently required by plumbing regulations, especially for restaurants. In general, such interceptors have been used for one or for all of the following reasons: (1) To prevent clogging of waste lines with grease, (2) to prevent large quantities of grease from reaching the sewage disposal works, (3) to facilitate the reclaiming of grease because of its economic value. The latter reason is, of course, at present a very important one for intercepting all waste grease and fats. The separation of gasoline and oils from waste water is also accomplished by use of a similar type fixture installed in the plumbing system; this paper, however, will be concerned primarily with grease interceptors.

The grease interceptors used at present are for the most part commercial products of various patented designs constructed of cast-iron (ceramic models have been used during the war). If properly installed and serviced, they do a fair job of preventing fats and grease from getting into the sewerage system. Proper installation and servicing, however, is usually the exception. To perform its job properly an interceptor should be installed as close to the fixture discharging greasy wastes as possible, and should be so designed and installed as to be easily cleaned. The less mixing and emulsifying there is, the easier the grease will separate from the waste water. Also the possibility of clogging the drain lines between the fixtures and the interceptor will be prevented if the interceptor is installed near the fixture.

Up until a few years ago the use of grease interceptors, especially in domestic installations, has in general not been overly successful. The interceptors were too small to handle adequately the rate of flow, and the owners did not properly remove the grease which had been collected in the interceptor. If, however, it is desired to separate the grease from the waste water in as complete a manner as possible and also to have the grease in good condition, an interceptor of the proper size installed right at the fixture which discharges greasy waste water is the best solution to the whole problem of grease removal.

Design Principles

A great many types of commercial grease interceptors are and have been on the market and with them many "home-made" designs. The basic principle of grease interception, however, in all such designs is that of gravity-differential separation. That is, the liquid greases and fats separate from the waste water in the interceptor, when the velocity of flow is reduced, owing to the difference in specific gravity.

Gravity-differential separators for liquids are used in many commercial processes, such as, for instance, in the petroleum industry. Of course, so far as grease interceptors are concerned, the problem is complicated by the fact that rarely is it a case of separating pure grease from water. The presence of soaps and food particles increases the difficulty of separation. Of interest in connection with grease-interceptor design is the congealing temperatures and approximate specific gravity of various common fats listed in Table A.

TABLE A

Type of Fat	Congealing Temp. °F.	Specific Gravity
Butter Fat	76-67	0.91
Beef Tallow	88-85	0.90
Mutton Tallow	106-97	0.94
Pork Lard	86-81	0.91

If the interceptor is installed in a kitchen, the temperature will usually be between 80° F. and 90° F. In any case, the discharge of hot water through the interceptor will heat it up, and the time required for passage of greasy wastes through an interceptor will usually not permit congealing to occur. In other words, separation must occur with the grease in a liquid state. Because of the short time it takes for liquids to pass through, cooling interceptors by the use of water jackets is of no consequence, and such interceptors should not be installed because of the danger of having made a direct connection from the pure-water lines to the interceptor, thus producing a potentially dangerous cross-connection.

Since grease separation is due to gravity-differential, a quantitative analysis of what occurs as waste water flows through an interceptor may lead to the establishment of some basic design data. For simplicity let us assume that pure grease and water enter near the bottom of a rectangular-shaped interceptor L feet long, B feet wide, and with a water depth of D feet. The interceptor will do a good job of separation if, as the flow goes through the interceptor, the mean velocity of flow is such as to permit a grease globule to rise a vertical distance D feet in a length of L feet. If we neglect, for a moment, the presence of turbulence we see that the controlling item in the sizing of the interceptor for any particular rate of flow is the rate of rise of the grease globules. If the size of the globules is known, the maximum velocity of rise can be calculated readily from known principles of fluid mechanics, by equating the buoyant force on a globule to the force of fluid resistance. An expression for the terminal velocity of a spherical grease globule is as follows:

$$v = \sqrt{\frac{4d(w - w')g}{3Cw}} \qquad \text{(Eq. 1)}$$

where d is the diameter of the globe, w and w' the unit weights of water and grease respectively, and C a fluid-resistance coefficient, which in texts on fluid mechanics is given as a function of the Reynolds number, which for the globule is vd/ν where ν is the kinematic viscosity of the water, and v the velocity of rise of the globule.

Obviously, the size of the grease globules is an important item. Observations and certain analyses indicate that the design of interceptors should be based on separating out globules of about 0.05 cm. The rate of rise of globules much less than this size is so small that gravitational separation is impracticable, and globules much larger than this will be easily separated. Taking a grease having a specific gravity of 0.90 and the water temperature as 150° F., the rate of rise (v in Eq. 1) of a 0.05 cm. globule is about 0.05 ft. per sec. For a globule of this size the time to rise a vertical distance D is $D/0.05$, and this should be equal to the time of flow for a distance L, which can be expressed as L/V, where V is the mean velocity of flow through the interceptor. We thus have:

$$V = \frac{0.05L}{D} \qquad \text{(Eq. 2)}$$

In most commercial interceptors of rectangular shape, the length is from 1.5 to 2 times the water depth. In that case, according to Eq. (2), the mean water velocity through the interceptor can thus be about 0.08 to 0.10 ft. per sec. Because it is impossible to secure uniform velocity distribution, however, and owing to the presence of some turbulence, the design value of V should not be over one-half of that given by Eq. (2). Modern commercial interceptors having good efficiency check this value of V fairly well.

In all interceptors, the separated grease accumulates in the top of the device and must be removed manually. Naturally, as the grease accumulates, the water-flow cross section decreases and the efficiency of the interceptor soon begins to drop. Some type of baffling is usually necessary and helpful, especially near the inlet. Such an inlet baffle ordinarily has louvers which distribute the flow and give it a gentle upward motion. Many baffles in the interceptor body, however, are undesirable since they induce turbulence. The outlet opening should, of course, be near the bottom at a point farthest from the inlet. Provision should be made for relieving air from the top of the interceptor which is brought down by the incoming flow. In general, to have settleable solids accumulate in a grease interceptor is undesirable. Our plumbing drainage and sewerage systems are designed to carry solids and there is no need of collecting them in a grease interceptor if it can be prevented.

Probably one of the most important design features of a grease interceptor is provision for easy cleaning and removing of accumulated grease. If this is not done, the chances of the interceptor's being properly serviced become remote. The interceptor cover should be light and easily removable, and the grease-accumulation chamber should be free of baffles and partitions which would get in the way during cleaning.

Sizes and Installation

The determining item, so far as grease interceptor size is concerned, is the rate of flow from the fixture or fixtures to which the interceptor is connected. This, of course, requires a

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knowledge of the rate of flow from various types of plumbing fixtures discharging greasy wastes. If several fixtures are connected to the same interceptor, then some estimate must be made of the probable simultaneous discharge.

Considerable data have been collected on the discharge rate from various fixtures. In Table B is given the recommended minimum rate of flow capacity of any interceptor connected to the fixtures shown:

TABLE B

Type of Fixtures	Rate of Flow in G.P.M.
Small Residence or Apartment Sink	5.0
Large Residence Sink or Dishwasher	10.0
Restaurant Kitchen Sink	15.0
Single Compartment Scullery Sink	20.0
Double Compartment Scullery Sink	25.0
2 Single Compartment Sinks	25.0
2 Double Compartment Sinks	35.0
Dishwashers for Restaurants:	
Up to 30 gal. water capacity	15.0
30 to 50 gal. water capacity	25.0
50 to 100 gal. water capacity	40.0

In addition to its rate-of-flow capacity, a grease interceptor should be rated as to its accumulated-grease capacity. This is the amount of grease in pounds that the interceptor can hold before its average efficiency drops below, say, 90 per cent.

The grease interceptor should be installed, as mentioned before, as near the fixtures as is practical, and its top should be easily accessible for cleaning. The discharge pipe from the interceptor should *always* be vented so as to prevent siphonage of the interceptor contents. This is quite important.

Testing and Rating

In order that interceptors may be properly rated as to capacity, a standard laboratory test is necessary. After years of experience in testing various types of interceptors, investigators have found that various results can be obtained by slight variation of the test procedure. Though field conditions cannot be duplicated in the laboratory, nevertheless the test procedure should have some relation to what the interceptor will experience in the field. A great many test methods have been tried in order to isolate the important items which affect interceptor efficiency. Thus, it has been found that the type and degree of mixing of the liquid grease and the hot water before discharging into the interceptor is an important item. The temperature of the grease and water and grease concentration did not have much effect on the results.

About two years ago, the U. S. Army Engineers, Construction Branch, asked the Iowa Institute of Hydraulic Research to test and rate each type of interceptor that was to be installed in the army cantonments. At that time a standard test was developed which has been used extensively and which seems to give reliable and consistent results. The apparatus used in the tests is shown in Fig. 1.

The hot water for the tests is supplied from a 200-gallon gravity-feed tank which has water and steam connections for controlling water temperature. Below this tank is a calibrated "flow-control" valve, the setting of which determines the rate

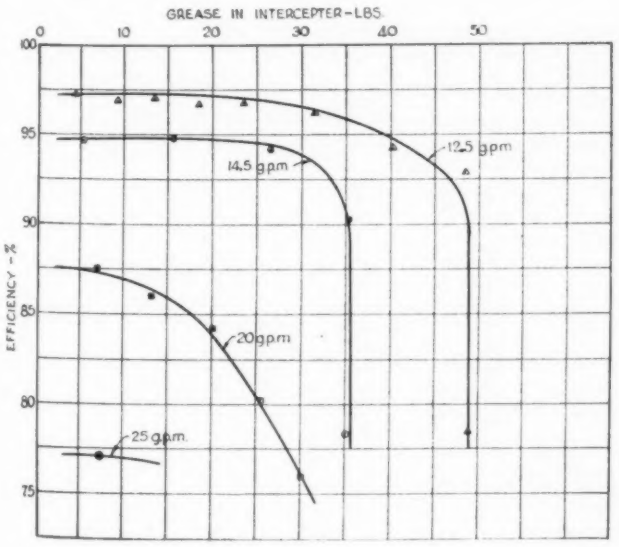


Fig. 2.—Typical laboratory test data for a commercial grease interceptor.

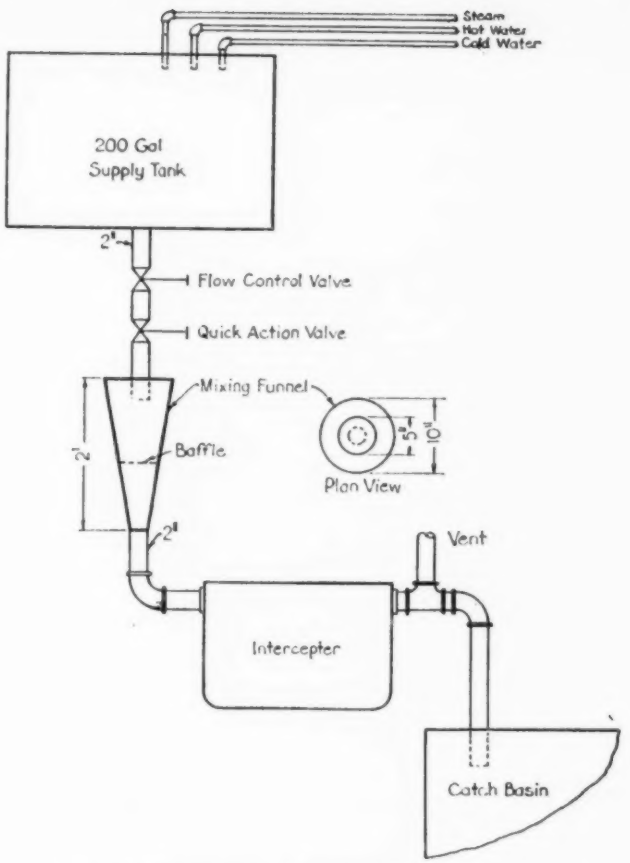


Fig. 1.—Arrangement of apparatus for testing grease interceptors in the laboratory.

of flow. Below this valve is a quick-action valve, which permits quick starting or stopping of the flow. The hot water from the tank discharges into a mixing funnel into which hot grease is poured at a uniform rate during the flow of hot water. The mixing obtained is thus not subject to any manual control. The mixture then discharges into the interceptor. Beyond the interceptor is a large tank from which the grease escaping the interceptor can be caught and skimmed off.

During the discharge of hot water, pure pork lard is added at a concentration of one pound for five gallons of water. This gives a grease concentration of about 2.4 per cent by weight. The water and lard are at a temperature of 150° F. Before starting a run the interceptor is filled with water at 150° F.

A run consists of discharging, at a selected rate of flow, the hot water and grease mixture at a number of two-minute periods. After each two-minute period the grease leaving the interceptor is collected, de-watered, and weighed. For certain large interceptors this collection of escaping grease is made only after two or three discharge periods. The discharging of the mixture of water and grease in two-minute periods is continued until the grease-retaining efficiency of the interceptor drops very appreciably. The average efficiency is calculated as follows:

$$\text{Efficiency} = \frac{(G_t)\ 100}{G_i} \qquad \text{(Eq. 3)}$$

G_i = Total grease in pounds added to interceptor since start of run.

G_t = The pounds of grease actually intercepted; this also equals G_i minus the total grease in pounds that left the interceptor since the start of run.

The foregoing value of efficiency is plotted against total grease in the interceptor, G_t. Thus it is possible to obtain the average grease retaining efficiency of any interceptor from the time it is clean to when a certain amount of grease is accumulated. A typical plotting of test data is shown in Fig. 2.

Authorities have agreed that the average efficiency for the rate of flow at which an interceptor is rated and for its specified grease-retaining capacity should not drop below 90 per cent. Also,

it appeared desirable to have some relation between rate-of-flow and grease-retaining capacity. It appeared reasonable to have the grease-retaining capacity in pounds be at least equal to twice the rate-of-flow rating in gpm. Thus, an interceptor rated at 25 gpm. must be able to retain at least 90 per cent of the grease discharged into it, in accordance with the above described test procedure, until it holds at least 50 lb. of grease. The majority of grease-interceptor manufacturers rate their products in accordance with the above method of testing and rating.

General Comments

Reclamation of grease as near its source as possible is unquestionably the ideal solution to the problem of grease removal from sewage; however, many practical difficulties hinder the working out of this solution in practice. Thus, for instance, the general use of grease interceptors on residential kitchen sinks is of debatable significance. The small amounts of grease usually discharged in kitchen sinks are so mixed with soaps, washing compounds and food wastes that gravity separation in an interceptor is practically impossible. Of course, residential plumbing drains do get clogged with grease, which means, of course, that in certain instances large quantities of pure grease, not emulsified with soap, etc., are discharged into home sinks. In these cases interceptors would be advantageous.

Of course, in the case of restaurant and commercial-type kitchens, the problem is different. Here, inasmuch as large quantities of pure grease and fats are discharged into the sinks, interceptors should always be installed on such fixtures. If, however, the important problem is that of reclaiming grease of high economic value, then it appears that a somewhat different arrangement might be desirable for establishments which have significant amounts of waste grease. In such cases the better procedure would seem to be to have a special fixture available which would be used only for discharging greasy wastes. In other words, all water containing grease and fats would be discharged into a special "separator" which would intercept the relatively pure grease and discharge the water to the sewer. No wash water or soap solutions would be discharged into this fixture. By following this plan much more and much better grease could be salvaged. On a small scale such a method might be used even in homes. However, the success of such grease-salvaging procedures would depend entirely on the public co-operation, and we question whether the American people are sufficiently imbued at present with the need for grease salvaging to carry out any such procedures very extensively.

From certain information which we have been able to obtain, it appears that large quantities of grease are being collected from the interceptors installed in army camp kitchens. Since very few data on actual field operation of grease interceptors are available, it is hoped further definite information about the operation of these interceptors will be obtained as time goes on. These interceptors are of supposedly proper size, and if orders are obeyed they will receive frequent cleaning. Data on their functioning should give an indication as to whether our laboratory testing and rating method is sound. There are good indications that it is, but as always the final proof is "How does it actually perform in practice?"

EXPANSION CHART*

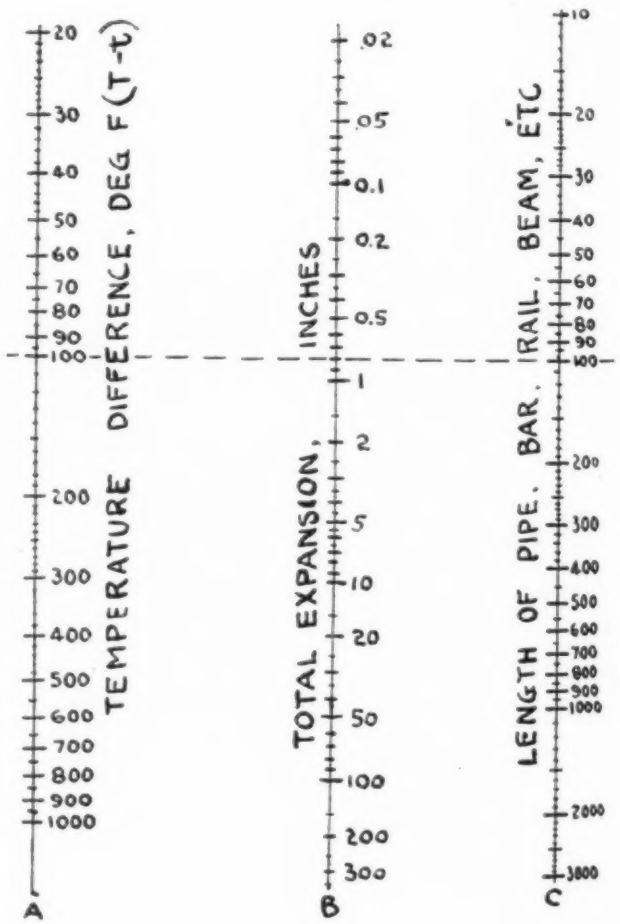
This chart is useful for determining the expansion in piping, rods, tubes, beams, shafts, columns, struts, etc., for any ordinary temperature difference. It makes "longhand figuring" unnecessary. Simply lay a straight-edge across the chart once or stretch a fine black thread across, as indicated by the dotted line, and the problem is solved.

Example: If a certain pipe line 100 ft. long is subjected to a temperature variation of 100 deg. F., what will be the total expansion in inches?

Simply connect the 100 deg. *temperature difference*, column A, with the 100 ft. length, column C. The intersection with column B gives the answer as 0.8 in. That is all there is to it.

The chart can also be used for determining the allowable temperature difference where a definite amount of space is available for expansion and contraction. Thus if the length of the pipe is 100 ft. and the allowable expansion is 0.8 in. the same line would show that a temperature difference of 100 deg. F. would be the limiting amount.

*From an article by W. F. Schaphorst, M.E., Newark, N. J., in WATER WORKS & SEWERAGE, March, 1939.



SEWAGE TREATMENT PLANT OPERATION

Compiled from Various Sources

By A. P. BANTA

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A DAILY inspection of each major unit of the plant by the plant operator should be made, any unusual conditions recorded, and faulty conditions immediately remedied. The plant operating data should be fully compiled each day, and kept, as an aid in maintaining the plant in proper operating condition. Such records are valuable for future reference and as evidence.

It is especially important that the daily flow or rate of flow of sewage reaching the plant be measured and recorded. Methods commonly employed to determine sewage flow are as follows:

1. Weir measurements.
2. Counters on dosing tanks of known capacity.
3. Capacity of pumps and period of operation.
4. Recording meters (Venturi, weir, etc.).
5. Dosing tank period for filling.

Records of power consumption and supplies, tools or chemicals obtained should be maintained. A daily log book for keeping such records and information concerning general or unusual conditions is very useful.

The air temperature, wind direction and general weather conditions should be recorded daily. This information is valuable in the investigation of nuisance complaints such as odors.

It is also essential that a few simple tests be made to determine the efficiency of separate units of the plant or of the plant as a whole. The results of these tests and a record on operation will:

1. Indicate to the operator whether the plant is or is not functioning properly.
2. Aid the municipality or institution in case of law suits resulting from alleged pollution or nuisance.
3. Assist consulting engineers in planning changes or additions to the plant.
4. Enable the officials responsible for the operation of the sewage disposal plant to prepare reports to the public and to the State Department of Health on the operation and efficiency of the plant.

The operator should have for ready reference plans or blue print designs of the plant, showing the dimensions of each unit and all pipes, valves, gates, etc., in order that he may be able to fully understand the construction and operating details of the plant.

A competent sanitary engineer can materially improve operating conditions through guidance and advice. It would be advisable for municipal or other officials to retain consulting engineers, or an experienced operator from a nearby plant, for a few years after completion of a new plant in order to supervise plant operation. Operating difficulties may frequently be prevented or readily corrected by following their advice.

Operators of sewage treatment plants should always remember that sewage gases are combustible and also explosive under certain conditions. Such gases may be toxic and asphyxiating. It is, therefore, necessary to take unusual precautions where such gas may be present either in the sewer system or at the treatment plant. No open flames or other sources of ignition should be permitted. Adequate ventilation of manholes and other similar enclosures before entering is always advisable. Moreover, no one should ever enter a place where such gases are likely to be present without an emergency rope attached to his person and two watchful attendants stationed outside. When the gases are toxis, i. e., containing poisonous gases, a special gas mask should also be used.

The following are a few fundamental details of the proper operation of sewage treatment plants. Local conditions may alter somewhat the following operating details but in general these notes give a resume of the operating procedure for the various types of treatment described.

Grit Chambers

1. Grit chambers should be cleaned after every large storm.
2. Grit chambers should be cleaned whenever tests show that the grit compartment (the compartment below the inverts of entering and outlet channels) is filled with hard material to 50 or 60 per cent of its capacity.
3. The grit should be promptly disposed of by burying it or drying it on sludge beds. If it is sufficiently clean, it may be used for filling in low land.

Coarse Screens (1/2-Inch to 3-Inch Openings)

Screenings, unless properly disposed of, are frequently a serious source of nuisance at a treatment plant because they decompose readily, giving off obnoxious odors. They will also encourage the breeding of rats and flies unless properly cared for. Furthermore they are unsightly and repulsive to visitors.

1. Remove the screenings twice daily or oftener if necessary to prevent clogging of the screen and consequent backing up of sewage in the outfall sewer.
2. Promptly dispose of screenings by:
   (a) Burial, covering by at least 12 inches of earth, either by ploughing under the soil or shallow trenching. They may be placed in trenches, treated with borax, chloride of lime, or other disinfectant and covered with earth as early as possible.
   (b) Incineration where possible.
   (c) Grinding, either returning ground material to the sewage flow or mixing it with the primary tank sludge going into digesters.
3. Clean the screen chamber, removing all accumulated sludge and grease, at least once a day.

Fine Screens (1/2-Inch or Less)

1. Fine screens are usually continuously mechanically cleaned, either by water, air or revolving brushes, and therefore all moving parts should be properly lubricated to keep them in proper working condition.
2. Remove and dispose of screenings promptly by:
   (a) Mixing with garbage and incinerated.
   (b) Dewatering by pressing, and then incinerating or burying the solids.
   (c) Burial, covering by at least 12 inches of earth, either by ploughing under the soil or shallow trenching. They may be placed in trenches, treated with chloride of lime, or other disinfectant, and covered with earth as early as possible.
   (d) Digested with other solids removed by sedimentation.

Septic Tanks

1. Sludge and scum should be removed whenever they accumulate to a combined depth equal to 1/4 the depth of the tank below the flow line.
2. Sludge should be removed when large suspended particles are observed in the effluent, or when the daily test of settleable solids in the Imhoff cones shows any appreciable increase of settleable solids in the effluent.
3. All inlet and outlet channels, pipes, weirs, valves, etc., should be kept clean and free from accumulations of grease and grit.

Mechanically Cleaned and Plain Settling Tanks with Separate Sludge Digestion

(A) Plain Settling Tanks:

1. The settled solids should be completely removed at frequent intervals, at least twice daily, from the settling tanks to the separate sludge digestion tanks. Mechanical equipment installed for the collection of sludge should be operated for a sufficient length of time before drawing sludge to insure complete removal of the deposited solids.
2. Septic action in the plain settling tanks as evidenced by "gassing" or rising sludge or an appreciable increase of settleable solids in the settling tank effluent should be avoided by prompt and regular removal of sludge to the digestion tanks.

3. Grease and scum should be removed at least daily, using the mechanical skimming device or by using a perforated hand-skimmer and the tanks should be maintained in a clean appearing condition at all times.

(B) Separate Sludge Digestion Tanks:

1. The contents of single tanks should be stirred or agitated at intervals by recirculation or other means to assure proper "seeding" of the fresh solids and to afford rapid and efficient digestion. Such stirring also lessens the chance of acid production, "foaming" difficulties, and excessive scum formation. Where there is surplus capacity or stage digestion this operation is of doubtful benefit.
2. Where heating coils or other adequate means of heating the contents are available, the temperature should be maintained at about 85° F.
3. The amount of fresh solids added daily should not be over one twentieth (5%) of the amount of "ripe" sludge in the tank, on the basis of the dry solid content of each.
4. Care must be used in removing digested sludge to avoid the removal of such a large amount that the ratio of "ripe" sludge to fresh sludge added drops below twenty to one.
5. By concentration tanks or otherwise the solids content of the sludge added to digesters should be as high as is practical to avoid loading the digesters with liquid which reduces the time of the solids in the digesters, creates undue disturbance and chills the contents. This matter is far more important than is generally realized.

Imhoff Tanks

1. Sedimentation Compartment:

(a) Grease and scum in the sedimentation compartment should be removed daily. This is best accomplished where no mechanical means is provided, by the use of a dish-shaped perforated skimmer. Such grease and scum should be promptly disposed of by burying or burning. Where objectionable conditions are not likely to develop, the grease and scum may be placed in the gas vents.

(b) The sides and slopes of the sedimentation compartment should be carefully scraped with a rubber squeegee and all solids pushed down through the slots at least twice a week.

(c) The slot in the bottom of the sedimentation compartment must be kept open and free from obstruction of any kind. At least once a week the slot should be cleaned by the use of a heavy chain drag, taking care to traverse the entire length of the slot.

(d) Failure to carefully follow the foregoing operating procedure will usually result in an effluent of inferior quality and where a trickling filter is used there will be deposits and accumulation of solid material on or in such filter, due to grease, scum or sludge being carried through the tank.

(e) Where the design of the tank will permit, the direction of flow through the sedimentation compartment should be reversed at least once a month, in order to distribute the solids uniformly in the digestion or sludge compartment.

2. Gas Vents or Scum Compartment:

The scum in the gas vent should be broken up at least weekly to afford an easy escape for the gas from the digestion or sludge compartment. Control of the scum may be accomplished by some of the following treatments:

(a) Breaking the scum up with a hoe, rake, or other suitable tool.

(b) Hosing the scum in the gas vents with water under pressure.

(c) Punching holes through the scum at two-foot intervals with a wooden pole.

(d) The scum may be removed and dried in a sludge drying bed if its depth approaches 2 to 3 feet in the vents. Usually it burns readily and if incinerated may create a hazardous flash, due to grease content.

(e) Treatment of the scum in the gas vents daily with hydrated lime at a rate of about 10 lbs. per 1,000 population has been found to be helpful in controlling excessive scum formation and to aid in maintaining the pH value of the digesting sludge.

3. Sludge Compartment:

(a) The height of the sludge in the sludge compartment should be determined at inlet and outlet ends of the tank at least once a month. The use of a pump for this purpose is the most desirable and satisfactory. The use of the plate or disc method is not usually satisfactory. The following are suitable methods for measuring the depth of sludge:

(1) One method involves the use of a pitcher pump provided with a rubber suction hose, weighted on the end and the length marked on the hose at intervals of 2 feet, measuring from the weighted end toward the pump. The hose is gradually lowered through the slot in the sedimentation compartment, meanwhile constantly pumping, and the length of immersed hose, when sludge first comes through the pump is determined. When the sludge elevation is reached, the pump will usually "choke" before sludge appears.

(2) The sludge depth may also be determined by use of an iron plate or weighted wooden block, about 12-18 inches square, attached to a wire or chain lowered through the gas vent. The plate or block will stop when the sludge is reached and the distance from the surface to the sludge level is determined from the graduated wire or chain by which the device is lowered. The pump method is by far the most positive procedure.

(b) Sludge should be removed:

(1) Whenever the sludge depth approaches within 18 inches of the slot in the sedimentation compartment.

(2) In small amounts at frequent intervals rather than in large amounts at longer intervals. Usually some sludge should be drawn every month.

(c) Sludge should be removed at a slow regular rate to avoid the formation of a channel through the sludge in the sludge compartment and the withdrawing of partially digested sludge or the liquid above the sludge.

(d) If sludge does not flow readily, it may be started by:

(1) Agitating with water under pressure through perforated water pipes in the bottom of the sludge compartment, if such pipes are available.

(2) Applying water pressure through hose immersed into the sludge compartment or through the sludge riser pipe.

(3) Agitating around sludge inlet with long rods through the sludge riser pipe.

(e) Avoid withdrawal of all the sludge from the tank. Leave sufficient digesting or "seed" sludge to prevent the difficulties incidental to starting a new or completely cleaned tank. Usually not over half the depth of sludge in the tank should be removed at any one time.

(f) After use the sludge pipes should be flushed out and refilled with water or sewage to prevent the sludge from hardening and clogging the sludge pipes.

4. "Foaming:"

There are a number of possible causes for "foaming" such as industrial wastes like milk or cannery wastes, "mash" or "wine" wastes, drawing out too much sludge at one time, or increased temperatures in the sludge compartment. Foaming is almost invariably associated with an "acid" condition of the sludge and in such cases may be minimized or corrected by treatment to counteract the acidity. However, there is such a thing as alkaline foaming but it is not common.

When foaming occurs, there are a few simple treatments which may under certain conditions remedy or improve the condition.

(a) Relief may sometimes be obtained by drawing some sludge.

(b) Hosing the gas vent area with water under pressure will sometimes help.

(c) Cutting the tank out of service if possible and allowing it to rest will permanently improve conditions.

(d) Paddling the foam with long-handled hoes is also temporarily effective.

(e) Hydrated lime added through the gas vents at daily intervals will frequently correct a foaming condition.

(f) Treatment of the raw sewage with chlorine at rates of 20 to 50 pounds per million gallons will often bring foaming under control.

Trickling or Sprinkling Filters

1. The nozzles should be inspected daily. All clogged nozzles should be cleaned and any broken nozzles replaced.
2. The surface of the filter bed should be kept free from plant growths.

3. The formation of ponds or "pooling" on the surface of the nlter bed may be remedied or prevented by:
   (a) Flushing the surface of the filter with a fire hose.
   (b) Raking or forking the surface of the filter bed.
   (c) Punching holes through the top layer of the filter medium with iron bars.
   (d) Heavy applications of chlorine, chlorinated lime or other chemicals for short periods applied directly to the bed surface or in the filter bed influent at the dosing tanks. The treatment is usually most effective and economical if applied to the low night flows of weak sewage.
   (e) Operation of units may sometimes be improved by cutting them out of service occasionally for a period of 12-48 hours.
4. Where the design of the filter bed will permit, the presence of large numbers of filter flies may be controlled by periodic flooding of the filter beds. Chlorine application, as for ponding, is also very helpful but not a 100% effective remedy. In flooding the sewage it should be chlorinated to delay oxygen depletion and septic action during the holding period.
5. The distribution piping should be flushed periodically.
6. The underdrains should be flushed out occasionally with water hose or other suitable flushing devices, if the filters are so constructed as to make this possible.
7. Where revolving distributors are installed, frequent inspection of water or mercury seals should be made
8. Where revolving distributors are installed, the flow from the nozzles should be adjusted so that the flow delivered is proportional to the area covered, i. e., the end nozzle should deliver twice as much as the one half-way out from the center. The center nozzles should deliver very little flow—only thin share for the ratio of area covered.

Secondary or Final Settling Tanks

1. The sludge should be removed frequently to prevent septic action and gasification of the solids which results in discharge of solids with the final effluent. This cleaning is usually necessary at least once a month, or oftener, during the summer season.
2. The depth of the sludge should be determined at frequent intervals by the use of a sludge sounder or other device.
3. The sludge may be disposed of in the same manner as sludge from primary settling tanks, i. e., pumped into primary settling tanks or into separate sludge digestion tanks direct for digestion. In the latter case density of the sludge is an important consideration to perclude the effects of dilution on the digesters.

Sludge Drying Beds

1. Sludge beds should not be filled to a depth of more than 12 inches or to such a depth that the sludge cake cannot be removed within two weeks in good drying weather.
2. The surface of the drying bed should be kept clean and free from all previously discharged sludge. *Never discharge wet sludge upon dried or partially dried sludge.*
3. The surface of the sludge drying beds should be kept nearly level to afford an even distribution of sludge over the surface of the bed.
4. Dried sludge may be used for fill, buried, or spread upon the ground, where it will not be carried into a stream or watercourse by rain.
5. In many communities the sludge is readily disposed of by giving or selling it to nearby farmers for use as fertilizer. This should be encouraged as sewage sludge has been found to have fertility and soil improving values. It is not advisable, however, to use it for the fertilization of crops which are served in an uncooked condition. For best results the sludge cake should be shredded or pulverized before attempting its sale.

Sand Filters

1. The beds should not be used continuously but sufficient time should be allowed between doses for the bed to drain thoroughly and "rest" between applications. Satisfactory operation of a filter bed depends on the presence of oxygen in the bed and therefore opportunity for the entrance of air into the bed must be provided by the "rest" periods.
2. The beds should be periodically cut out of service, allowed to dry and the surface raked or scraped clean.
3. "Pooling" should not be allowed to develop on the beds as this tends to produce septic action, obnoxious odors and an effluent of poor quality. Pooling indicates that cleaning is necessary. Frequently a harrowing is all that is required. Eventually skimming will be required.
4. The surface of the beds should be kept level to afford uniform distribution of the sewage.
5. Weeds, grass, etc., should not be allowed to grow on the beds.
6. The depth of the bed should be maintained by removing during cleaning only the least possible amount of sand. Once a year clean sand should be added to replace that removed during the year.
7. Where natural sand percolation beds are used it is sometimes advisable to shallow harrow the beds after careful removal of all organic deposits.

Chlorination

The purpose of chlorination is to prevent bacterial contamination of streams or other bodies of water, used for municipal or industrial water supplies, irrigation, shell-fish growing, recreation or bathing.

The effective chlorination of sewage or sewage effluents depends upon the presence of adequate quantities of chlorine at *all* times in all portions of the sewage for sufficient time to provide the necessary period of contact between the disinfectant and the bacteria in order that disinfection may be efficient.

The strength of any sewage varies greatly with seasonal changes, with the days of the week and during the hours of the day. The amount of chlorine required varies accordingly.

1. Chlorine should be applied in such quantity that there will be a residual of at least 0.5 p.p.m., as determined by the ortho tolidine test, after 15 minutes contact of chlorine with the sewage or the effluent, depending upon whether pre- or post-chlorination is practiced.
2. The test for residual chlorine should always be made at the time when the sewage or effluent is the strongest or when it requires the largest amount of chlorine for treatment.
3. Weigh chlorine tank each day at the same time to determine the amount of chlorine used during the preceding 24-hour period.
4. The chlorine must be applied *continuously* in the proper quantities.
5. Before connecting a new chlorine cylinder to the control apparatus, blow off some of the chlorine from the new cylinder into the air in order to prevent clogging of the apparatus. Store the empty cylinders apart from the full ones so as to avoid mistaking empty cylinders for full ones.
6. Examine the chlorine apparatus daily and if any trouble is found which cannot be corrected by the operator, notify your superior at once.
7. It is not advisable to draw chlorine from any one cylinder at rates higher than 40 lbs. of chlorine in a 24-hour period, because of the danger of "freezing" and slowing up the chlorine flow. If more chlorine is necessary, two or more containers should be connected and used in parallel.
8. The operator should always have on hand a stock of the usual replaceable parts for the chlorine apparatus and a reserve supply of chlorine cylinders.
9. Leaks in the chlorine apparatus should be promptly repaired to prevent corrosion of the equipment and structures. Such leaks may be detected by the formation of white fumes when tested in the immediate vicinity with the vapors from an ammonia solution. (Household ammonia is satisfactory).
10. During cold weather heating facilities should be provided to maintain a temperature of 60° F. and never less than 50° F., in order to prevent so-called "freezing" in the chlorinator and stoppage of the chlorine flow. It is frequently necesssary to heat the water supply to the tray of the chlorinator to prevent "chlorine-ice" formation.
11. Chlorine is a liquid when under pressure in the cylinders and is an irritating gas which is heavier than air when released. It may produce serious after effects if inhaled. A satisfactory gas mask should be available at all plants where chlorine is handled.

Activated Sludge Tanks

1. Sludge percentage tests (settling for ½ hour a liter graduate) should be run twice daily on the mixed liquor of the aeration tanks, sampled at the half-way point.
2. When the sludge percentage of the mixed liquor exceeds 30% (300 c.c. in the liter graduate), sludge should be withdrawn. Some plants have been found by experience to operate more efficiently at a lower per cent.
3. Excess sludge should be wasted into the raw sewage if primary sedimentation is practiced or wasted directly to digesters or via thickener tanks if these are provided. Such tanks are an important adjunct to the functioning of digesters.
4. Direct drying of untreated sludge results in odor and fly nuisance.
5. For small plants, dissolved oxygen tests on the effluent

from samples taken from the clarifiers should be run daily. B. O. D. and suspended solids tests on the effluent should be run at least twice weekly.

6. Cracked or leaking air diffusion plates should be watched for and quickly repaired.
7. Air and sewage regulating valves should be operated at least once a month to insure against sticking.
8. Bulking is said to occur when unusual amounts of suspended solids pass out with the effluent. It may sometimes be controlled by one of the following practices:
   (a) Increasing the supplied air or reducing the solids carried in the aeration units.
   (b) Continuously adding lime sufficient to bring the pH up to 9.0 (for small plants).
   (c) Adding small amounts of fine clay (for small plants).
   (d) Moderate chlorination of the returned sludge.
   (e) Bypassing the sewage until condition is corrected.
   (f) Eliminating industrial wastes from the sewerage system when these are proven to be causative.

If bulking becomes a chronic problem, the administrative authority should be fully informed and an investigation by a competent sanitary engineer recommended.

9. Air pressure on this diffuser manifold should be recorded regularly, as increase in pressure results in increased power cost.
10. Operation of final settling tanks is similar to operation of primary settling tanks.

Neatness, Cleanliness, Beautification

1. It is very easy for a sewage plant operator to become negligent. The appearance of his plant reflects in most cases his own character.

2. A limited planting of shrubs and trees will add greatly to the appearance of a plant. Most administrative authorities, if properly approached, will provide at least in a small way, for this improvement. Select hardy plants which require a minimum of attention.

3. Paint is an investment. A plant where painting is neglected will soon require expensive replacements. Learn all you can about paint; many conditions require special paint.

4. A set of neat, well-kept records is a convincing argument when an operator is asking for improvements.

5. A clean plant is usually a well-run plant.

6. An untidy plant is a hazard to life and limb. Slippery steps, unused equipment left in the way, can be the cause of a nasty fall.

7. Work around sewage is always subject to health hazard, extra care and cleanliness are, therefore, essential. A first-aid kit should be on hand and used for even minor injuries. Arrange for prompt medical service for more serious accidents.

8. Make a study of the life cycle of flies and mosquitoes. A knowledge of this subject will enable an operator to control with a little thought and effort, most insect nuisances.

Last but Not Least

9. Join and take an active part in your local sewage works association. The journals and the exchange of operating experience with other members is a rich source of progressive ideas.

Acknowledgement—This paper constituted a lecture in the Water and Sewage Tanks Operators Short Course of the 11th Annual Institute of Government at the University of Southern California, Los Angeles

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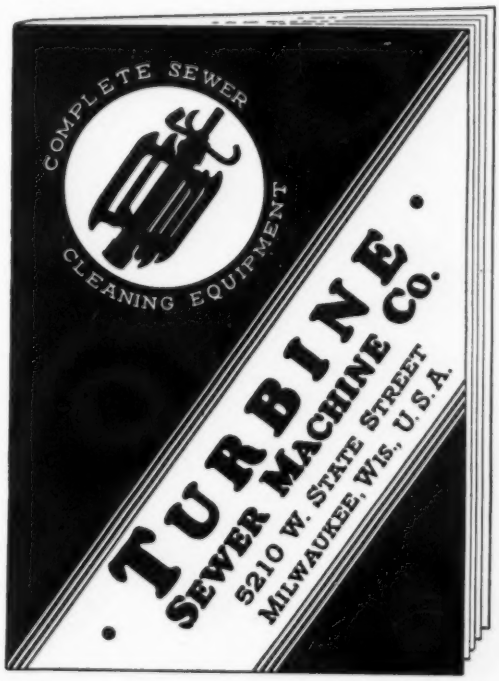

Pin a Medal on the Engineer who said,

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SOME SEWAGE WORKS OPERATING KINKS

By A. M. RAWN*
Editorial Associate, Water & Sewage Works

SEWER cleaning operations on a large trunk sewer system give rise to many notions and ideas regarding best methods. The writer presents a few operation and maintenance "kinks" which have been evolved entirely by the field operation forces on the Los Angeles County Sanitation Districts' system. Nothing new is claimed for the ideas and many an operation man may recognize in each, or all, a counterpart of what he daily practices.

Sewer Cleaning

The writer in the October, 1935, issue of WATER WORKS AND SEWERAGE described the use of rubber beach balls for cleaning lines up to 30 inches in diameter. Above 30 inches the balls are expensive and hard to obtain so that for cleaning circular sewers larger than 30 inches in diameter the district forces use a form of "hoe" shown as Fig. 1. It is simply and easily made and can be assembled in the ordinary manhole without much trouble.

In operation the "hoe" is assembled in the manhole and the proper sized flexible belting is attached to its downstream face. Belting is cut and fitted for each size of pipe from 30 inches to 60 inches for circular pipe, and is properly stiffened with sheet metal as is required in the larger sizes. With the exception of the belting attached to the face of the "hoe," the same equipment is used for cleaning all sizes from 30 to 60 inch pipe lines.

Following its assembly in the manhole, the "hoe" is thrust into the downstream pipe and the trail line is attached to the tail board holding the weights. The sewage backing up behind the "hoe" propels it through the pipe, moving any accumulated debris ahead to the manhole below.

The "hoe" is surprisingly efficient in cleaning the sides and invert of the line. Care must be taken to secure a reasonably close fit between the pipe and the outer edges of the belting so that the belting will have to flex forward to permit the escape of water collected behind the "hoe" face. If this precaution is taken, and the tail board is suitably weighted, one may be assured that just about anything movable below the backed-up water line will be shoved or sluiced ahead to the next manhole. Like the pneumatic cleaning ball, described in the October, 1935, issue of WATER WORKS AND SEWERAGE, the "hoe" moves easily over obstructions.

Root Cutting

Many types of root cutters have been manufactured and sold. A number have been tried on the districts' lines. None of them, however, has proven much more efficient than the simple, inexpensive device shown herein as Fig. 2. It is simply and easily made of ordinary butt-welded steel pipe of commercial sizes.

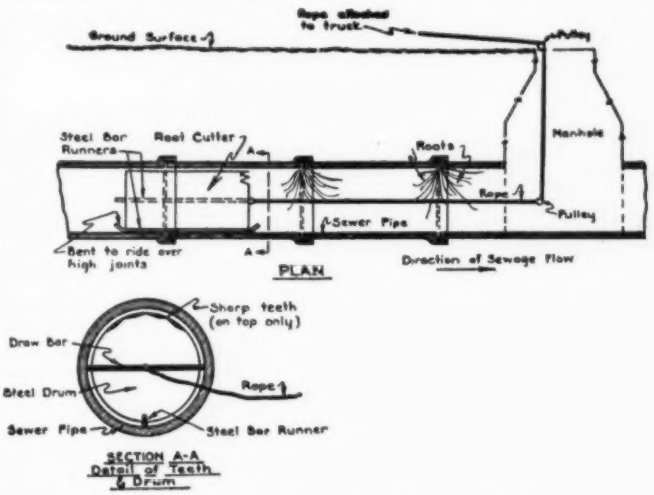

Fig. 2.—Root Cutter for Sewers of 12 to 36 In. Diameter.

*Chief Engineer, Los Angeles County Sanitation Districts, Los Angeles.

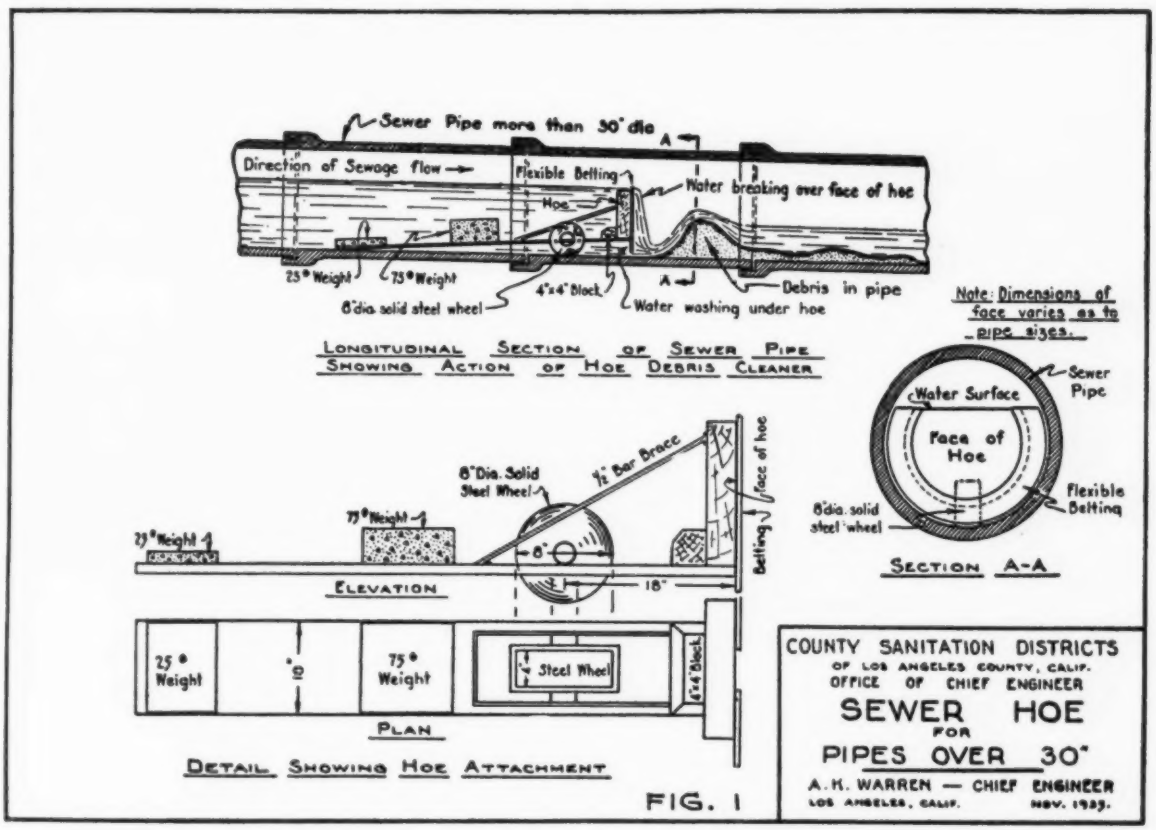

Fig. 1.—The Sewer "Hoe" for Cleaning Sewers Larger Than 30 In. Diameter.

To make it, take a piece of pipe about 18 inches long, serrate and sharpen the edges of both ends, and weld four shoes to the outside of the pipe. Affix halters for the pull and trail lines—and the job is done.

For an 8-inch line job use a 6-inch diameter pipe and ¼-inch strap runners or shoes. For a 12-inch line, use 10-inch pipe. A 12-inch diameter pipe is large enough for a 15-inch line, and so on. The sharpened notches in the pipe ends should be rather deep and wide so that they may pinch and hold the root, tearing or cutting it loose as its point of entrance into the line at which point it is usually quite small and relatively weak.

The pull line is thrust or pulled through with sewer rods, if it cannot be floated through the line. Pulley-blocks are then suitably arranged at the downstream manhole so that pulling may be done with the maintenance truck.

Lacking the proper diameter pipe for an operation, the operation and maintenance crew once manufactured a similar root cutter from a discarded water tank salvaged from a dump heap. It is still used upon occasion.

Air Relief Valve

Fig. 3 illustrates a type of air relief valve used in the 3½-mile (36 in. diam.) effluent pump line leading from the districts' sewage treatment plant. The valve pictured is installed at a summit in the line about 2,000 ft. from the plant where air must be tapped off if the line is to operate efficiently. It is also important that none of the effluent escapes through the blow-off.

A number of valves were tried but all operated with indifferent success until that sketched was installed. As may be seen, it consists of a simple float operating a steel ball valve attached to the end of a section of ½-inch pipe. The ball seats in a ground valve seat tapered to catch and center the ball as it rises. The valve case is constructed from a 5-inch diameter, butt-welded, steel pipe.

The secret of the mechanism is the small hole drilled in the ½-inch pipe near the ball. This is to permit admission of air into the light metal float, equalizing the pressures inside and out on the float and preventing collapse of the thin metal cylinder. When first constructed the hole was omitted and the float promptly collapsed, rendering the valve ineffective. When rebuilt, the air inlet hole was drilled and the valve, operating about three times a minute, has been in continuous service for more than six months.

It may be thought that condensation or spray might eventually fill the float cylinder with water, and the writer admits having shared such a belief at one time. Convinced, however, by the plant chief operator who designed and built the valve, that if the float did eventually fill it could be easily drained and put back into service, the installation was permitted. Now it has been proven that the principle is sound. No water escapes through the valve.

Grease Collection and Incineration

At the district's treatment plant, grease is skimmed manually from the tanks and burned. For a number of years skimmings were accumulated in galvanized iron cans and transported to drainage beds. When drained they were picked up in wheelbarrows and conveyed to the incinerator.

Handling the galvanized iron cans was a dirty and expensive item of labor as was the spreading on drainage beds and the reaccumulation and transportation to the incinerator. The grease

Wheel Barrow Used for Transporting Skimmings Containers to Incinerator.

draining beds were odorous in spite of all precautions and the incinerator ash was about 90 per cent sand—the latter being picked up with the grease from the beds.

The chief operator requested and received permission to construct the containers and transporting barrow.

The container or pan may be made any convenient size. That shown holds about 3 cubic feet. Placed in the bottom of the pan and standing on a rigid framework which elevates it an inch off the pan bottom, is a ¼-inch square mesh screen, the purpose of which is to provide drainage for the skimmings. The pan is so constructed that all drainage is toward the 1½-inch pipe shown in the right front of the pan at the base. Drainage from the skimmings goes directly back into the sewage tank.

The pan is filled with skimmings as they are taken from the sewage and permitted to drain for a couple of hours before being taken to the incinerator. Twelve such pans are in use

For transporting the pans to the incinerator the pneumatic tire wheel-barrow (illustrated) was constructed and is used. The operator wheels up to the high side of the pan, steps across it and engages the forward cross brace in the two forward pan dogs, then upon lifting the wheel-barrow handles, the rod welded across the back of the pan catches in the two swinging hooks

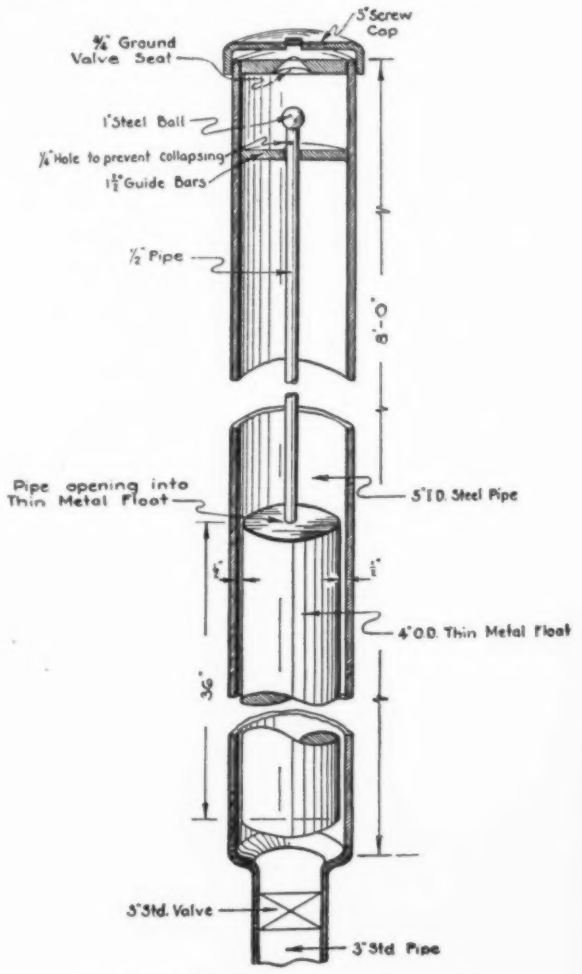

Fig. 3.—Air Relief Valve for Force Main.

attached to the barrow handles, and the load is securely engaged. The penumatic tire prevents wear on concrete walls and cross walks; makes for easy rolling.

It may be seen that when the pan is lifted the slope of the pan bottom is reversed and any water shaken out of the screenings during transportation stays in the pan until it is set down.

At the incinerator a movable sheet metal storage pan is located in front of the charging door. The operator swings the loaded wheel-barrow handles over the storage pan, engages the pan's back rod in the two forward dogs in the storage pan, releases his load and then tips the screenings into the storage pan with a hook.

The pans are cleaned daily with hot water and once a week with grease solvent. The pans and the wheelbarrow were all built in the plant shop by the plant operators.

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7. Smaller treatment plants. Because Transite Sewer Pipe minimizes infiltration, plant capacity is more efficiently used. Where new plants are being designed, substantial savings in initial cost of construction and equipment may often be effected.

*Reg. U. S. Pat. Off.

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Fig. 2959
Aeration tank with diffusers operating. One raised for cleaning.

Swing Diffusers—Controlled aeration for the Activated Sludge Process. For towns of 4,000 and up. Diffuser tube assembly can be raised to tank walk. Simple regulation of air supply from tank walk. Tube cleaning without dewatering tanks or interrupting operation.

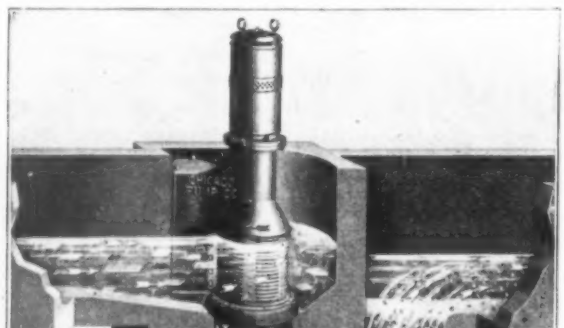


Fig. 2202—Comminutor
Eliminates screenings and screenings disposal—No burial; No incineration. Cuts coarse sewage matter into small settleable solids without removal from the channel. Ask for Bulletin 185.

Fig. 2504—Vertical Open Shaft Non-Clog Pump.
Universal Flexible Couplings in shaft compensate for misalignment. Ask Bulletin 126.

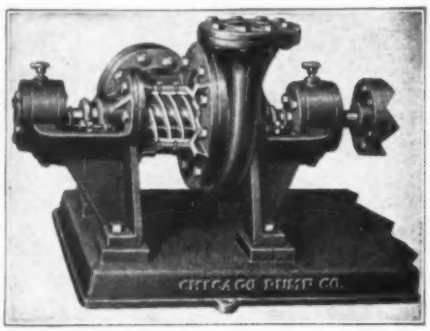
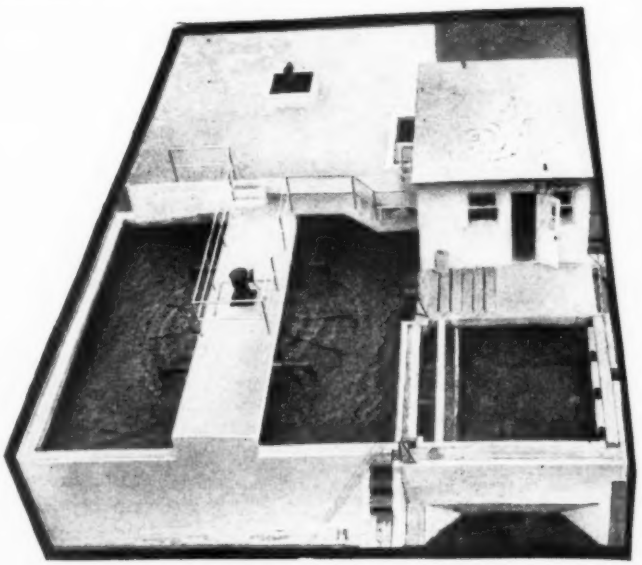


Fig. 1860—Scru-Peller Primary Sludge Pump
Screw conveyor in the suction ahead of the impeller. Stellited cutters prevent clogging. The capacity can be varied instantly. Ask for Bulletin 190.



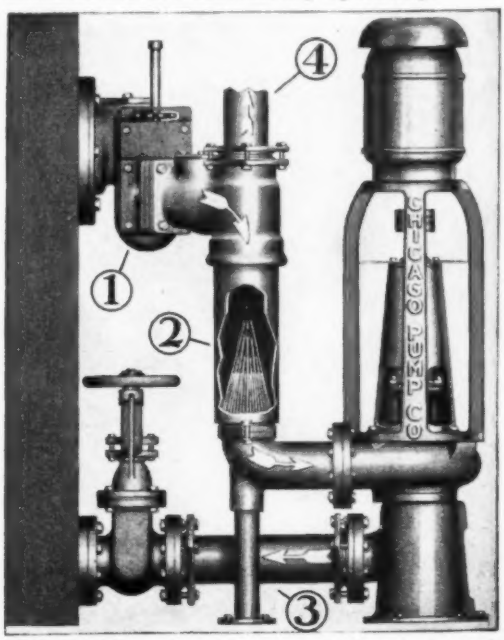
Chicago "Pakage" Sewage plants are ideal for small communities, industrial plants, airports, housing developments and institutions because they were specifically developed for small populations of 100 to 5000. Properly designed these plants can handle industrial, cannery and other wastes as well as the usual community sewage.

They require a minimum of operating supervision and produce a sparkling clear effluent. As they are free from flies, foul odors and unsightly appearance, location near dwellings is entirely feasible.

Operator training by Chicago Operating Sanitary Engineers is provided with each plant. Local operators without previous sewage treatment experience operate existing plants and perform other municipal duties.

Ingenious automatic features of Chicago "Pakage" Plants simplify operation and assure successful performance. Aeration and clarification are performed in a single tank with positive, automatic sludge control. One sludge setting covers a wide range of sewage flows and strengths.

Initial cost of "Pakage" plants is low and they are inexpensive to operate. Since 1934 over 100 of these plants have been installed and are successfully operating.



The FLUSH-KLEEN pump cannot clog because it is designed so that no solids can reach the impeller. Sewage flows through inlet pipe (1). Solids are retained on screen (2). (3) Strained sewage flows through idle pump to wetwell. As pump switches on, Checkvalve closes, sewage on strainer is backwashed and carried off through outlet pipe (4).

OPERATING FUNDAMENTALS OF THE ACTIVATED SLUDGE PROCESS

T. R. HASELTINE, *Cons. Engr.,*
The Chester Engrs., Pittsburgh, Pa.

THE properties of activated sludge which are most important to the plant operator are, first, its clarifying or absorptive powers, and, second, its oxidizing power.

When activated sludge is mixed with sewage it absorbs the finely divided suspended matter, the colloids, and probably much material in true solution in or on its floc surfaces. If the sludge is then allowed to settle out, the supernatant liquor will be clear instead of cloudy. This clarification action is very rapid and is probably completed in 10 to 30 minutes.

The oxidizing power of activated sludge is generally assumed to be entirely due to biological action. Although its oxidizing action may be some 60 to 130 times faster than the normal oxidizing rates, as measured by the standard B.O.D. test, it is still much slower than adsorption or clarification.[1] The highest rate of oxidation usually occurs when the sludge first comes in contact with the sewage. It then falls off gradually for the next two to five hours, after which it continues at a more or less uniform rate for considerable periods of time.

In any properly functioning plant there is a balance between these two properties. Almost all of the troubles encountered with the activated sludge process may be traced to a lack of proper balance. For example, if biological oxidation proceeds more rapidly than adsorption, the sludge floc become very small and compact. This dense floc does not produce a sparkling clear effluent although nitrification may be very high. Furthermore, some of the floc is apparently broken up into minute particles which do not settle in the final clarifiers, even though the main body of the sludge settles very rapidly. On the other hand, if oxidation lags behind adsorption, the sludge floc will become so light that it will not settle in the final tanks and the sludge will lose much of its clarifying power. Since adsorption is very rapid and almost automatic, the operator's problem is to regulate the rate of biological oxidation so that it just keeps pace with adsorption under the aeration periods prevailing in his particular plant.

There are several conditions under which oxidation will progress more rapidly than adsorption. Probably the most common is over aeration. It is encountered in plants having excessive blower capacity in a few large constant speed machines. It is best overcome by carrying more sludge in the aeration tanks. A second, but less common, cause of over oxidation is too long an aeration period. The best corrective is to bypass part of the aeration tanks. If that cannot be done, then increase the amount of sludge in the mixed liquor. A third cause of over oxidation may be lack of sufficient "food" in the sewage. The writer encountered such a condition at Grove City during the early spring. At such times our sewage flow is increased by ground water to four or five times its dry weather volume. Most of the excess flow has to be bypassed and hence only one-third to one-fourth as much organic matter enters the aerators as normally. We have found that the admittance (after liming) of digester supernatant to the aerators was quite helpful at that season.

Causes of Oxidation Lag

There are at least six possible causes of oxidation lagging behind adsorption, as follows:

(1) *Inadequate Oxygen Supply:* In any aeration tank (mechanical or diffused air) we have two opposing forces, one, the aerator device injecting air into the mixed liquor, and two, the biological organisms using up oxygen in their life processes. Dissolved oxygen determinations on the mixed liquor give a measure of the excess of the oxygen supply over the oxygen consumption. It is absolutely essential that the rate of supply be at least equal to the rate of consumption, but there is nothing to indicate that any great excess is beneficial.[2] If there is no dissolved oxygen in parts of the aeration tanks then either the oxygen supply must be increased by using more air, or the rate of oxygen consumption must be cut down by reducing the amount of sludge or sewage in the mixed liquor. Naturally sludge reduction is the most general practice.

(2) *Germicidal Wastes:* Since oxidation is entirely biological it is evident that germicidal wastes will stop or retard it. If the germicidal action is due to excessive acidity or alkalinity neutralization with lime or acid will eliminate it. (The optimum pH for biological oxidation is 7.0 to 7.5 although there is little retardation at pH 6.4 to 8.0.[4] If it is due to phenolic compounds or other toxic substances, the only certain remedy is to exclude them from the sewers. In some cases, however, where the concentration of the poison is quite uniform and not too high it may be possible for the sludge organisms to become accustomed to it.

(3) *Excessive Carbon Dioxide:* Carbon dioxide is one of the principal end products of biological oxidation[3,4] and excessive amounts of it are known to be toxic to bacterial activity. Sewage already contains relatively large amounts of CO_2. Although aeration is an efficient means of removing carbon dioxide the rate of its production during the activated sludge process is so great that the net reduction due to aeration may be slight. In such cases it is quite possible that the rate of oxidation is limited by the CO_2 content of the mixed liquor. When such conditions prevail it is logical to apply lime to the inlets of the aeration tanks.[4] Either one of two general schemes may be followed: (a) daily applications of 50 to 300 lbs. of lime per M.G. of daily flow applied in one to four "shots" during hours of peak sewage flow, or (b) continuous lime applications at rates of 300 to 500 ppm. until such time as the pH at the outlet of the aerators approaches 8.0. The best way to tell whether or not excessive CO_2 is causing trouble is to try the lime treatment. It is fortunate that this treatment can do no harm so long as the pH at the outlet of the aerators does not exceed 8.8, whether or not it does any good.

(4) *Septic Sewage or Sludge:* Septic action not only increases the carbon dioxide but produces other products that retard biological oxidation. Avoid sludge accumulations in sewers, channels, primary or final clarifiers. The return of large volumes of poor quality digester overflow liquor to primary tanks may cause the entire sewage flow to become septic. Frequent sewer flushing may be helpful. Prechlorination at various points along the collection system may be even more helpful. Partial chlorination of the return activated sludge has proved beneficial at some plants but chlorine is a powerful germicide and particular care is required to avoid over-chlorination of the sludge which might destroy its oxidizing power.

(5) *Excessive Grease Content of the Sludge:* If the grease content of the sludge is excessive the sludge particles become coated with a film of grease thus preventing intimate contact with oxygen, even though tests show an abundance of dissolved oxygen in the mixed liquor, hence oxidation is greatly retarded. Wiest's[6] work indicates that the activated sludge process functions satisfactorily when the grease content of the sludge is 5 to 7 per cent, but gives considerable trouble when the grease content is around 20 per cent. The writer's experience indicates that 13 per cent grease in the sludge makes for difficult operation while 30 per cent grease causes the sludge to lose its clarifying power and a very inferior effluent results. If the grease content of the sludge is generally excessive then some means of further removing grease from the incoming sewage is indicated. But where the trouble is caused only by periodic shots of mineral oil the writer has found a 25 per cent increase in air for a day following such shots to be helpful.

(6) *Inadequate Aeration Period:* Even under ideal conditions biological oxidation is much slower than adsorption and, hence, there is some minimum aeration period required below which the process will not function. Usually there is nothing the operator can do but bypass part of the sewage flow and ask for more aeration tanks. In some cases, however, short circuiting in diffused air plants has been a factor that could be corrected by proper baffling. Current meter readings at various points may be helpful in designing or locating baffles. In other cases it has been possible to delay major extensions by utilizing part of the aeration tanks for sludge reaeration, thus lengthening the available time for biological oxidation.

Since both the quantity and strength of the sewage varies from hour to hour and day to day, it might be asked, how is the operator to maintain a constant balance between adsorption and oxidation at all times? He can not! In the normal plant oxidation probably falls behind adsorption during the high flow of strong daytime sewage and catches up during the low flow of

weaker night sewage. In some plants it is even possible to observe a gradual deterioration of the sludge through the week with recovery over Sundays. It must always be kept in mind that it is only prolonged periods of excessive adsorption that cause serious sludge deterioration, and that several days of excessive oxidation are required for its recovery. The closer the balance is maintained the lower will be the power costs, but the higher will be the laboratory and attendance costs. Therefore more laboratory work is justified in large plants than in small, but some is required in all.

Sewage Input vs. Air Input, and the D. O. Test

Since most if not all of the oxidation occurs in, or on, the sludge floc, it should be evident that most of the oxygen is consumed by the sludge. Therefore, it is a serious mistake to attempt to regulate the air input in proportion to the sewage flow. Dissolved oxygen determinations throughout the aeration tanks are the best criterion known for air regulation. If the air is to be varied throughout the day then these tests should be made at various hours throughout the day. If a constant rate of aeration is to be used then the tests should be made at whatever time the dissolved oxygen is at a minimum; this will usually be found a few hours *after* receipt of the peak sewage flow. Samples should be collected from various points between the inlet and the outlet of the aerators. Unfortunately the dissolved oxygen determination, the most important of all operating control tests, can not be made directly on the mixed liquor because of interference from the sludge floc. The accurate determination is extremely difficult and requires special equipment,[7] however tests for routine plant control may be made on samples of supernatant liquor siphoned off from larger samples of mixed liquor collected from the aerators. At Salinas, California,[5] the writer collected 2 liter samples from the tanks and allowed them to stand until sufficient clear water could be siphoned off to fill the small D.O. bottles. Ordinarily 5 to 15 minutes standing sufficed although at times of severe bulking as much as 40 minutes was required. That procedure is open to the objection that the sludge consumes some oxygen during the sedimentation period. However, on several occasions we siphoned off a second sample an hour or two after taking the first one and repeated the D.O. determinations; the decrease in dissolved oxygen in the supernatant over the elapsed time was surprisingly small. Theroux, Eldridge and Mallman[8] recommended the addition of 10 ml. of 10 per cent copper sulphate solution to the 2 liter samples of mixed liquor. The copper sulfate acts as a coagulant, speeding up sedimentation, and at the same time halts biological oxygen consumption. Theoretically, at least, this method is open to the objection that the copper sulfate tends to react with the potassium iodide added later liberating iodine and thus giving results that are too high. Heukelekian uses about half as much 10 per cent sulfuric acid in place of the copper sulfate. The writer has tried all three methods simultaneously at Grove City on several occasions and found little or no practical difference. We use copper sulfate in our routine work.

Regardless of the method used to separate out the sludge floc the Rideal-Stewart modification of the Winkler method should always be used for determining the oxygen in the small, siphoned samples because of the possible presence of nitrites in the liquor, see "Standard Methods."[25]

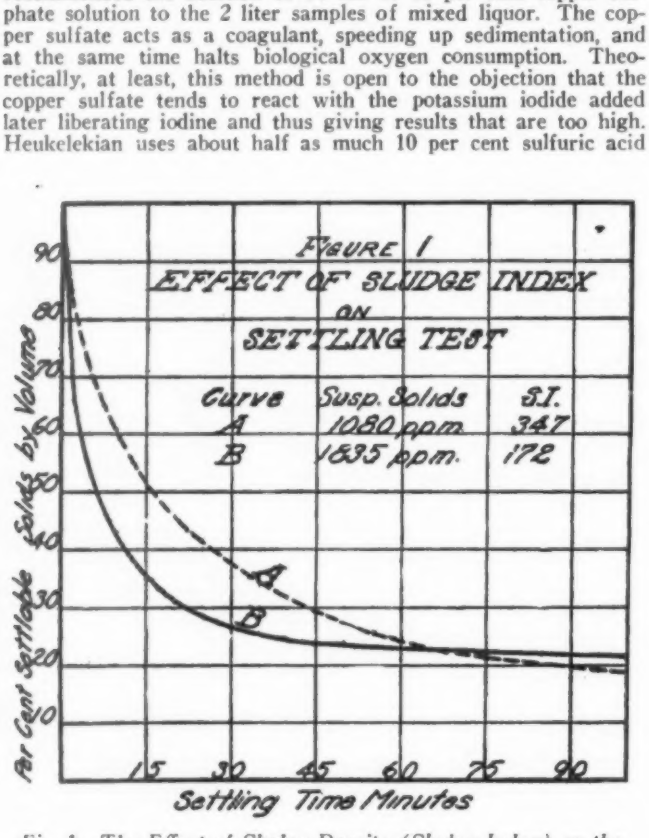


Fig. 1—The Effect of Sludge Density (Sludge Index) on the Volumetric Sludge Settling Test.

Measuring Sludge in Mixed Liquor

We have mentioned increasing or decreasing the amount of sludge in the aerators as a means of controlling the oxydation-adsorption balance. This leads us to the second most important control test, namely, some means of measuring the amount of sludge in the mixed liquor. By amount of sludge we mean *weight* of sludge solids. No universal rule can be given for determining the proper amount. Perhaps 600 to 1500 ppm. of suspended solids in the mixed liquor would cover most mechanical aerators while 1200 to 3000 would cover most compressed air plants. The higher the sludge content of the mixed liquor the faster will adsorption and oxidation progress, but the greater will be the power requirements. If the aeration period is short an increased sludge content will give a higher degree of treatment, but if an adequate aeration period is available almost as good purification can be obtained with a low sludge content and a considerable saving in power may be realized. With a low sludge content organic matter adsorption per unit weight of sludge is very high and hence a very high rate of biological oxidation must be maintained. Therefore, if the sludge content is low, there is more danger of a plant being upset by sudden shots of strong sewage or organic trade wastes.

Standard suspended solids determinations are the only accurate means of finding the actual weight of sludge solids in the mixed liquor. Unfortunately the test requires considerable time and, for small plants, expensive equipment. In spite of these handicaps the test is extremely valuable and should be made at least once a day at all but the very smallest plants. It should be made at the same time each day. The time selected should be such that the sewage flow for the past several hours has been nearly identical with that of the same hours of each preceding day—usually 8 or 9 a.m. will be the most satisfactory time. The writer has followed this practice at Salinas,[5] Topeka[26] and Grove City. We governed the daily amount of activated sludge, wasted almost entirely by the results of these determinations. This procedure does *not* accomplish the very desirable end of keeping the suspended solid content of the aerators constant at all times throughout the day. It was probably a minimum at the time the peak flow started coming in and at a maximum in the evening when the sewage flow drops off. However, it did serve to keep the *average daily* suspended solid content of the aerators fairly constant. If it is desired to hold the solid content of the aerators constant throughout the day then one of the approximate "short-cut" methods of solid determination (to be described later) may be used as frequently as desired, but the standard suspended solids test should be made daily as a check on the short-cut procedure. After considerable experimentation the writer has concluded that the saving in power made possible by hourly checks of the solid content of the aerators was not sufficient to justify the time consumed in checking, but such a conclusion can apply only to the plants with which he was connected. Anderson, employing a centrifugal estimation, has found the opposite to be the case at Rockville Center.[11, 12, 13]

Before leaving the subject of gravimetric solids determinations the writer wishes to make a plea for all operators to base their control on *suspended* solids determinations rather than on *total* solids determinations. It is the suspended solids in the sludge that exert the bulk of its oxygen demand. It would be permissible to base plant control on total solids determinations only if the dissolved solids were practically constant. Such is not the case—at Salinas[5] dissolved solids in the mixed liquor, and plant effluent, ranged from 600 to 1500 ppm. under varying sludge conditions; at Grove City the range is 200 to 500 ppm.

Because of the time required for suspended solids determinations various short-cuts have been used. The simplest are the settling tests and these were the earliest tests used. Samples of sludge or mixed liquor are simply collected in liter graduates, allowed to stand for some given time, and the per cent of settleable solids observed at the end of that time. In the early days it was usually customary to carry a volume of about 20 per cent sludge, after 30 minute standing of the mixed liquor. The test is still used and is quite helpful so long as its limitations are

kept in mind. Various standing times are used, ranging from 10 minutes to 1 hour or even longer.

The Sludge Index

Attempts to correlate the volume of settleable solids, as determined in this way, with the actual weight of suspended solids in the sample soon showed that no such relationship existed unless the sludge index[9] was also taken into consideration, see Figure 2. "The sludge index (S.I.) is the volume in cubic centimeters occupied by 1 gram of sludge solids after settling 30 minutes"; it is computed by multiplying the per cent sludge by volume after 30 minutes standing by 10,000 and dividing that product by the parts per million of suspended solids by weight in the same or a similar sample of mixed liquor. The longer the settling test samples are allowed to stand, i.e., the more the sludge compacts, the less the influence of the S.I. on the results of the test and, hence, the closer their approximation to the actual weight of sludge solids present. Figure 1 illustrates this point fairly well. In general the settling test alone cannot be relied on except in the smallest plants, and even then only in those small plants that are so amply designed that they are subject to little bulking. In such plants the possible saving in power may not be great enough to warrant the added cost of conducting more accurate determinations.

The Centrifuge Test

Another short-cut is the centrifugal test. This test consists of collecting samples of mixed liquor or sludge in 15 ml. centrifuge tubes and "spinning" them for a given short time (2 to 10 minutes) in an electrical centrifuge. A hand centrifuge will not serve because of the impossibility of always spinning the samples at the same speed.

The writer first heard of this test being used at the Indianapolis plant in 1928. Like the settling test, it is, at best, a volumetric determination. Nevertheless, it has two outstanding advantages over the gravity settling test. First, it is extremely rapid, and, second, it compacts the solids more and hence there is better correlation between its results and those of gravimetric suspended solids determinations. The writer has used a 2 minutes spinning period at 2600 r.p.m. at Salinas and Grove City. At both plants we found that the results of this test were greatly influenced by the sludge index, see Figure 3. Setter[10] used a 10 minute period at the same speed and got fair correlation with the suspended solids test so long as the suspended solids exceeded 1000 ppm.; at lower solids concentrations the volume of sediment was too small to permit satisfactory readings. Anderson[13, 11, 12] used a 5 minute period at 1800 r.p.m. at Rockville Center and considers the test very satisfactory although he gives no data to correlate it with suspended solids determinations. If the test is to be used the operator should first make sufficient trials to prepare a graph similar to Figure 3. Once such a graph is available he may determine the sludge index at daily to weekly periods and from that determination and the graph he can select the desirable centrifugal test results for more frequent determinations.

In addition to determining the amount of sludge being carried in the aerators it is desirable to know something of its condition. The sludge index, as previously described, is valuable in this respect. Color, odor, and floc structure are also valuable. Periodic microscopical examinations are useful. After a little experience these tests, or observations, will usually be found adequate for most plants. In some cases the volatile content or dissolved solids content of the sludge may be found helpful.[8] Still better tests required special equipment such as the Odeeometer,[14] Oxy-Utiometer,[15] etc.

Tapered Aeration vs. Delayed Loadings

It was previously stated that the rate of oxidation was usually highest when the sludge first comes in contact with the sewage and that afterwards it falls off gradually. If the air is supplied in just sufficient amounts to maintain a minimum dissolved oxygen content at all points in the tanks it will be found that considerably more air is used at their inlet ends than is used at their outlets. This method is known as *tapered aeration*. Since the rates of adsorption and oxidation increase with increased amounts of sludge, tapered aeration is more necessary when a high sludge content is carried. The rate of oxygenation is limited for any aeration device, but is generally lower for mechanical aerators than for compressed air plants,[17] hence the former usually carry less sludge and frequently provide longer aeration periods. At the Tallman's Island[18] plant provision is made for adding sewage or return sludge at several different points along the aeration tanks, thereby avoiding the necessity of using such high aeration rates at their inlets. In small plants it is possible to reduce the necessity for tapered aeration by returning a large volume of very dilute sludge. Experience has taught many operators that this practice is a good one for small plants in particular. The excess water reduces the oxygen demand of the mixed liquor at the aerator inlets and forces the sludge along the aeration tanks faster, thereby reducing the air requirements at the inlet end and increasing it further along the tanks.

Importance of Prompt Sludge Recirculation

It used to be considered good practice to carry a considerable depth of sludge in the final clarifiers in order to get as high a solid content, and hence as low a volume of return sludge as possible. Studies[19] have shown that the oxygen demand of such sludge is frequently so high that it is almost impossible to satisfy it by any aerating device. It has also been found[20] that the activity of the sludge was greatly impaired by even comparatively short concentration periods in the clarifiers and that the fresher the sludge the greater was its purifying power; the effect being most noticeable at plants having a short aeration period. Therefore, the rate of withdrawal from final tanks should be such as to remove the sludge as quickly as possible. In large plants where the cost of pumping excess water may exceed attendance, or in plants using sludge reaeration, it is wise to make frequent sludge soundings and to keep the rate of withdrawal at a minimum consistent with rapid sludge removal. In small plants that do not have sludge reaeration this extra attendance is not justified and a very high, constant, rate of sludge withdrawal can be used throughout the 24 hours. When the plant is treating a strong sewage the daily sludge withdrawal may be actually equal to the sewage flow with no ill results and, possibly, some benefit. (At first it might appear that such an increase in volume of sludge return would unduly shorten the aeration period; however, it may be shown mathematically that increasing the rate of return from 20 per cent to 50 per cent (equal sludge and sewage volume) cuts the nominal aeration period only 10 per cent because so much of the liquor in consequence passes through the aerators more than once.)

Most plants are not designed for sludge reaeration. If reaeration is used it is desirable to keep the rate of sludge withdrawal from the final tanks at a minimum, consistent with rapid sludge removal, in order to have the longest reaeration period, *providing it is possible to maintain some dissolved oxygen at all points in the reaeration tank*. If that can not be done then the rate of withdrawal *must* be increased so as to reduce the oxygen demand of the reaerating sludge.

Sludge Concentration Important

To intelligently divide the sludge withdrawal between return and waste it is necessary to make daily suspended solids tests on the mixed liquor, as previously discussed. If these results are

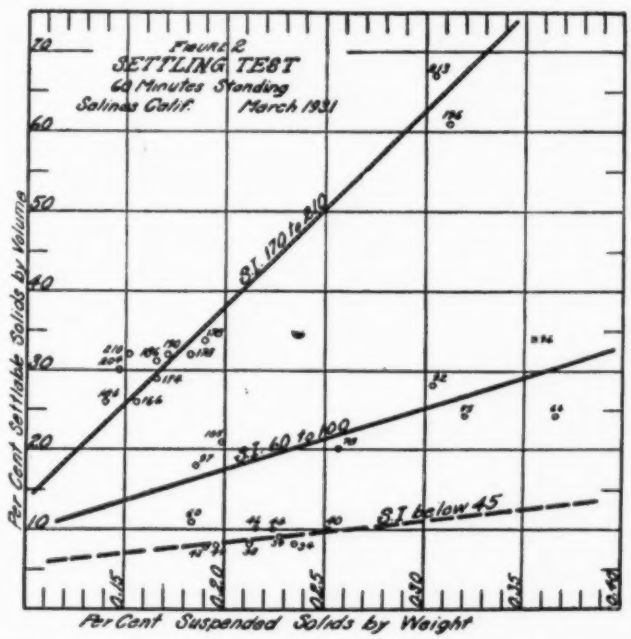


Fig. 2—A Picture of the Lack of Correlation Between Volumetric Sludge Test and the Gravimetric Test. The Necessity of Taking into Consideration the Density and Settleability (Sludge Index) Is Clearly Evident.

below the amount of sludge desired the amount going to waste should be decreased and vice-versa. Waste activated sludge should always be concentrated before introducing it into sludge digestion tanks. At most plants the only means of concentration provided is the introduction of waste activated sludge into the primary settlers. It has been the writer's experience that where this method is used better concentrations of primary sludge are obtained by wasting continuously over the 24 hours at a low rate than are obtained by wasting at a high rate for a shorter time. This method of wasting excess activated sludge has many disadvantages,[21] particularly when the sludge is bulked. A better, though less common method entails the use of a separate tank, termed a sludge thickener, and the use of chlorine.[22, 23] Where a thickener is provided best results are obtained by feeding it with mixed liquor direct from the aeration tanks instead of with sludge withdrawn from the final clarifiers. The clear, chlorinated supernatant from the thickener may be discharged to the incoming raw sewage, the return activated sludge or the final effluent. Either of the first two are preferable because of its beneficial action in overcoming septicity.

Troubles

Now just a few words concerning troubles. Sludge bulking is probably the most common trouble encountered with the activated sludge process. The term "bulking" has been used by some to cover the condition of sludge pouring over the weirs of final clarifiers, but the writer considers that an improper use of the term. A bulked sludge is simply a sludge that, after a given settling period, occupies a greater volume than would a normal sludge containing the same amount of suspended matter by weight. Bulking is best measured by the sludge index[9] as previously described. Whether or not a bulked sludge will pour over the weirs of final clarifiers depends, primarily, upon the design of those tanks. The more generous their design the higher can the sludge index become before the effluent is fouled.

It was previously stated that when oxidation lagged behind adsorption the sludge floc became so light that it would not settle properly, in other words, the sludge bulks. We have given six possible causes of oxidation lagging behind adsorption and suggested means of overcoming them. Any one of those six will cause bulking if not corrected. Certain microscopic, filamentous, growths will cause, or increase, bulking. Sphaerotilus is far the most common organism. Even though the sludge is constantly seeded with these organisms they do not flourish in the sludge if oxidation is keeping pace with adsorption, but if oxidation lags behind adsorption the sludge may become an ideal habitat for profuse growths. Correct the oxidation-adsorption balance and the number of Sphaerotilus will greatly diminish. Other filaments than Sphaerotilus may cause bulking, notably Cladothrix and Leptothrix, but their occurrence is so rare as to warrant no discussion here.

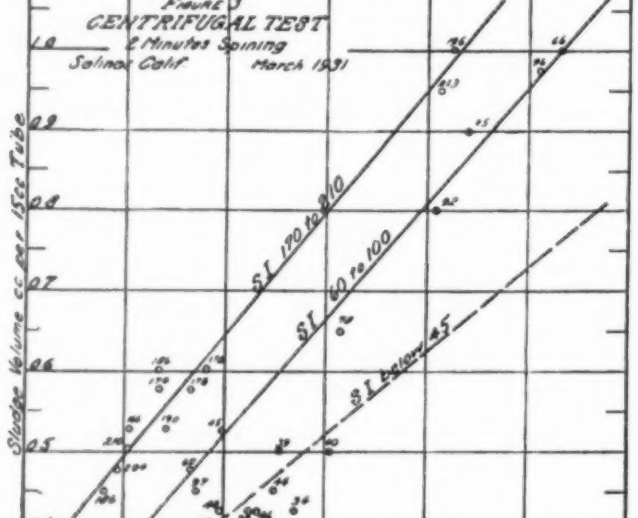


Fig. 3—A Picture of the Less but Evident Effect of Sludge Index Values on Suspended Solids Tests With the Centrifuge.

Bulking is not the only cause of excessive amounts of sludge passing out with the final effluent, although it is the most common one. There are two other causes. One is simply insufficient sludge withdrawal, the other is "sludge rising." Rising sludge is most commonly encountered in plants treating sewage high in nitrogenous matter (notably meat packing wastes) where nitrification is quite high. The sludge will settle rapidly in tanks or graduates, but will rise again in from 20 to 90 minutes. The sludge is lifted by accumulations of nitrogen gas. At some plants[24] it has been overcome by keeping the dissolved oxygen in the aerators very low; at others[16] it has been found helpful to increase the amount of sludge carried in the aerators.

Digester overflow liquor has caused considerable trouble at several activated sludge plants, among them Grove City. At most plants it is common practice to return digester overflow to the raw sewage, and, in a properly proportioned plant, the liquor will cause little or no difficulty providing its solid content does not exceed 0.2 per cent. If the overflow contains more than 0.5 per cent solids it turns primary clarifiers septic and covers them with gas-buoyed sludge. Under such conditions their effluent is very difficult to treat by the activated sludge process. At Grove City we have an overflow averaging well over 1.0 per cent solids. We add lime to it and turn it directly into the aeration tanks. The proper lime dosage, in our case, seems to be about 1 pound of lime to each 10 cubic feet of liquor. The liquor turns the aerators very dark, detracts from the appearance of the effluent, and unquestionably puts a tremendous burden upon the activated sludge process. However, we have been able to treat most of the liquor in this way although it seriously upset the plant when we attempted to return it to the raw sewage.

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For another thorough article on "The Operation of Activated Sludge Plants," by Dr. Gail P. Edwards, Chief of Laboratories, Dept. of Public Works, New York City, see page R-191, July 1947 issue of *Water & Sewage Works*.

EXPERIENCE IN DIFFUSED AIR ACTIVATED SLUDGE PLANT OPERATION*

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THIS review of activated sludge plant control practice is based on data and comments furnished by some 25 plant superintendents and chemists, who, even in these "questionnaire-able" days, found the time to set forth their procedures in a most detailed blank form. The complete co-operation of the following contributors is gratefully acknowledged:

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Ahrens, G. C., Superintendent, Omaha, Nebraska.
Allen, Wm. A., Superintendent, Pasadena, California.
Anderson, C. George, Superintendent, Rockville Center, N. Y.
Anderson, R. A., Superintendent, Muskegon Heights, Mich.
Barton, Ben H., Chief Operator, Findlay, Ohio.
Berg, E. J. M., Superintendent, San Antonio, Texas.
Bloodgood, Don E., Manager, Indianapolis, Indiana.
Bolenius, R. M., Chemist, Lancaster, Pennsylvania.
Brunner, Paul L., Chief Chemist, Fort Wayne, Indiana.
Collier, J. R., Superintendent, Elyria, Ohio.
Edwards, Gail P., Chief of Laboratories (Wards Island), New York City.
Froehde, F. C., City Engineer, Pomona, California.
Harris, R. C., Commissioner of Works, Toronto, Ont., Canada.
Henry, B. F., Superintendent, Pomona, California.
Larson, C. C., Chemist in Charge, Springfield, Illinois.
Lehmann, A. F., Superintendent, Hackensack, New Jersey.
Montgomery, J. R., Chemist, Pontiac, Michigan.
Munroe, E. H., Superintendent, York Township, Canada.
Philhower, Sara K., City Chemist, Gastonia, North Carolina.
Rhoads, Edward J., Superintendent, Lancaster, Pennsylvania.
Smith, E. E., General Superintendent, Lima, Ohio.
Turner, J. R., Superintendent, Mansfield, Ohio.
Wheeler, C. E., Jr., Engineer of Operation (Calumet Plant), Chicago, Illinois.

It is intended here to present data and comment on practical methods of controlling the principal operating variables, thus supplementing the elaborate committee report compiled by Langdon Pearse, chairman, and the sewage disposal committee of the American Public Health Association (see *Sewage Works Journal*, 14, 3, January, 1942). Only diffused-air activated sludge plants are represented in this study.

Insofar as the operator is concerned, there are five primary variables over which control can be exercised in an activated sludge plant.

1. Concentration of mixed liquor solids.
2. Rate of sludge return.
3. Rate of air application.
4. Aeration period.
5. Condition of the sludge.

These variables must be correlated into balance under the local conditions of load imposed upon each individual plant, and varied to meet seasonal and other fluctuations in load. Furthermore, the operator must view the variable factors as being interdependent, in the sense that any adjustment made in one of them may require manipulation of one or more of the others.

As a preliminary to the operation practices followed in the plants contributing to this summary, general data concerning each are given in Table I. It will be noted that a wide range in size, type, design and loading is represented.

Concentration of Mixed Liquor Solids

Most authorities regard control of the mixed liquor solids concentration, accomplished by apportionment of the activated sludge from the final settling tank between return and waste, as the most important of all operating variables. Certainly the recognition of this fact in recent years, with the resulting trend toward reduction of mixed liquor solids from the very high values formerly carried, has marked a notable advance in control practice.

Table II lists the average and range in mixed liquor solids carried in the aeration tanks of 21 plants. Thirteen of the plants reported show average concentrations of 1,500 to 2,500 p.p.m. to be used; at only three is the average less than 1,000 p.p.m. Four plants are also shown to average concentrations over 2,500 p.p.m., with the highest value (3,100 p.p.m.) at Lancaster, Pa. The variation in aeration solids carried, as indicated by the range in each plant, is interesting because the difficulty of close control is illustrated.

With the exception of one or two cases reported, it will be noted that lower sludge indexes are achieved where mixed liquor solids in the higher ranges are carried and that higher sludge indexes seem to accompany low mixed liquor solids values. This possibly indicates that sludge of better settling characteristics and higher density is usually obtained in the mixed liquor solids range of 2,000 to 3,000 p.p.m. than at lower concentrations, substantiating the general belief that more stable operation under shock loads is to be expected in the higher range.

At Calumet (Chicago), Wheeler endeavors to hold 2,500 to 3,000 p.p.m. of mixed liquor solids, although the upper limit is often exceeded during and following storms when the ash content of the sludge increases. It is considered desirable to hold the volatile solids content of the mixed liquor above 1,500 p.p.m. in this plant. Brunner at Fort Wayne is also primarily interested in the volatile content rather than the total suspended solids in the mixed liquor, reported as follows:

"Control of the mixed liquor solids is maintained at 1,500 p.p.m. volatile solids rather than any optimum value of suspended solids.

TABLE I.—GENERAL INFORMATION REGARDING CONTRIBUTING PLANTS

Plant	Average Flow (m.g.d.)	Average Primary Sedimentation Period (Hr.)	Aeration — Type	Aeration — Aeration (Hr.)	Average 5-Day B.O.D. Raw Sewage (p.p.m.)
Chicago (Calumet)	85	0.25	Spiral	4.0	93
Elyria, Ohio	2.5	1.67	Spiral	5.5	353
Findlay, Ohio	2.3	3.1	Spiral	5.5	405
Ft. Wayne, Ind.	17.6	1.3	Spiral	8.6	199
Hackensack, N. J.	3.2	3.2	Comb. (1)	12.0	200
Indianapolis, Ind.	53.3	None	Spiral	8.4	272
Lancaster, Pa. (North)	3.3	2.9	Spiral	13.7	209
Lima, Ohio	7.7	1.4	Spiral	4.8	134
Mansfield, Ohio	3.1	2.3	Comb. (1)	7.4	237
Muskegon Hts., Mich.	1.2	3.1	Comb. (1)	6.3	322
New York (Wards I.)	184	1.2	Spiral	4.5	167
North Toronto, Ont., Can.	7.3	2.0	Spiral	6.0	280
Omaha, Neb.	3.3	2.0	Sw.D. (2)	7.5	241
Pasadena, Calif.	9.5	None	Sp. & R.F. (3)	6.9	151
Pomona, Calif.	1.7	—	F.D. (4)	8.3	—
Pontiac, Mich.	4.2	0.9	Spiral	5.0	208
Rockville Centre, N. Y.	2.3	0.8	F.D. (4)	4.4	292
San Antonio, Tex.	22	0.5	Spiral	5.9	225
Springfield, Ill.	8	2.0	Spiral	8.7	165
Tenafly, N. J.	0.7	—	Spiral	7.3	144
York Twp., Can.	3.1	None	Spiral	4.3	—

(1) Combination diffused air and paddles.
(2) Swing diffusers.
(3) Both spiral and ridge-and-furrow tanks.
(4) Fixed diffuser tubes.

TABLE II—MIXED LIQUOR AND RETURN SLUDGE CONTROL

Plant	Mixed Liquor Solids (p.p.m.) Average	Mixed Liquor Solids (p.p.m.) Range	Return Sludge Av. Return Rate (%)	Return Sludge Av. Solids (p.p.m.)	Average Sludge Index
Chicago (Calumet)	2,850	1,720-3,870	22	15,700	45
Elyria, Ohio	2,270	700-3,600	29	11,600	52
Findlay, Ohio	1,500	500-4,000	26	5,000	150
Ft. Wayne, Ind.	2,320	1,400-4,100	27	9,700	111
Hackensack, N. J.	1,600	900-2,500	23	10,000	130
Indianapolis, Ind.	2,500	400-3,700	41	8,000	98
Lancaster, Pa. (North)	3,100	2,700-3,600	31	13,800	49
Lima, Ohio	1,720	1,140-3,110	33	7,200	92
Mansfield, Ohio	1,040	940-1,140	20	3,650	346
Muskegon Hts., Mich.	900	400-2,000	41	4,300	100
New York (Wards I.)	1,770	1,075-3,050	45	4,775	170
North Toronto, Ont.	2,300	1,500-3,500	19	13,500	93
Omaha, Neb.	1,685	1,240-2,300	45	4,320	74
Pasadena, Calif.	1,100	980-1,340	12	6,700	189
Pomona, Calif.	...	.....	...	....	...
Pontiac, Mich.	2,500	2,000-3,000	20	....	125
Rockville Centre, N. Y.	730	2,000-3,000	3.5	26,700	40
San Antonio, Tex.	1,500	.....	42	4,200	210
Springfield, Ill.	1,750	725-2,500	30	6,500	121
Tenafly, N. J.	2,340	2,120-2,670	34	12,550	103
York Twp., Toronto, Can.	570	370- 800	38	....	...

*A summary published in The Operator's Corner of *Sewage Works Journal*. Reprinted by permission.

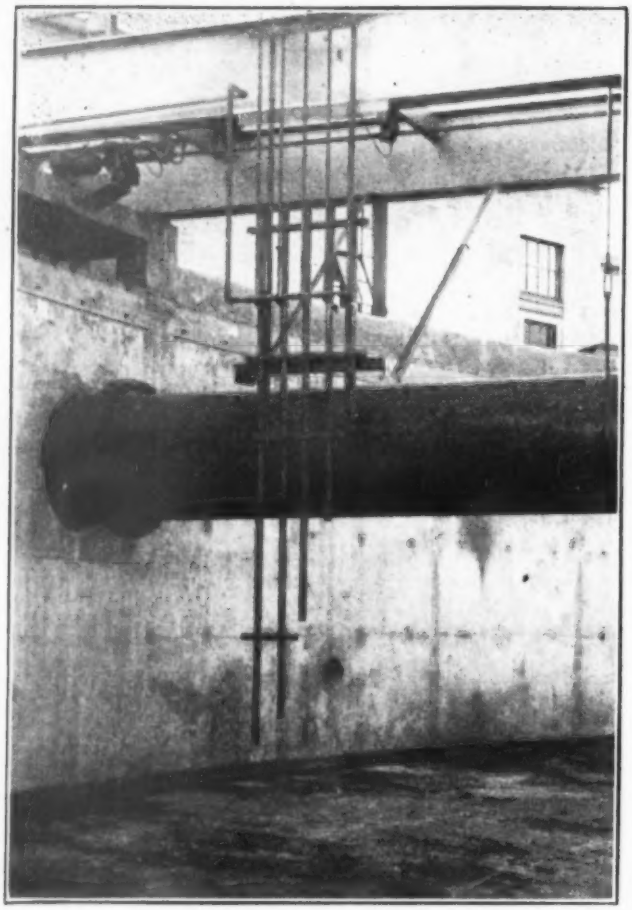

Fig. 1.—Air-lift bubbler for sludge blanket determination in final settling tanks at Fort Wayne, Ind.

It is necessary to run on this basis because of frequent slugs of river water which bring in fine silt and thus add weight to the solids. If we were to waste on the basis of a 2,000 p.p.m. suspended solids level during these periods, we would not have much active sludge in the mixed liquor."

At Indianapolis, Bloodgood found it desirable to carry the highest mixed liquor solids concentration possible with the amount of air available:

"It is our belief that more solids can do more work if adequately supplied with air. Every effort is made to keep the mixed liquor solids at a uniform concentration so as to eliminate the piling up of solids in any particular part of the plant. The solids are determined daily from composite samples and the amount of sludge to be wasted is gaged from the concentration found to be present."

Andersen at Rockville Centre, N. Y., at which plant the activated sludge treatment is followed by mechanical filtration of the effluent, defines the optimum mixed liquor concentration as "only that quantity (of solids) requisite to purify properly the sewage, while also insuring an excess of oxygen to maintain the health of the biological life." A very active sludge is present in this plant, averaging 88 per cent volatile solids. Operating conditions are also unusual in that an extremely dense return sludge (27,000 p.p.m. solids) prevails in normal operation. Under these conditions of balance, it has been possible to eliminate the effluent filtration for the past eight months with "the sludge index, settling of solids and sludge production stabilized to an apparently foolproof extent."

Larson (Springfield, Ill.) and Lehmann (Hackensack, N. J.) refer to the common need for adjustment of the mixed liquor solids to seasonal variations. Comparison of practice in these two plants, however, furnishes an excellent example of the effect of local conditions on plant control. Heavy loads received for treatment at Hackensack in summer months require about 2,000 p.p.m. of mixed liquor solids concentrations as against 1,600 p.p.m. in winter. At Springfield, higher volatile matter percentages in summer permit some reduction in the suspended solids carried. Turner (Mansfield, O.) checks occasionally for varying trends by trying lower and higher concentrations but thus far has always returned to a value of about 1,000 p.p.m.

Physical features of plant design may limit the flexibility of solids control, as at Muskegon Heights, Mich., where the air capacity and sludge return facilities are critical. Superintendent Anderson states:

"The solids concentration is governed by the load received from the primary tanks, dissolved oxygen in the final settling tanks in comparison with that in the last aeration tank, nitrate production and to a large extent upon our sludge withdrawal facilities. The latter have never permitted carrying more than 2,000 p.p.m."

Allen at Pasadena gives primary attention to the concentration of solids in the return sludge and finds that the mixed liquor solids concentration is controlled incidentally. His experience indicates 6,500 to 8,000 p.p.m. of return sludge solids result in best operation with about 1,200 p.p.m. of mixed liquor solids. Return sludge solids are estimated twice daily by centrifuge determinations.

Based on the experience represented here and some reference to the APHA committee report noted previously, the following summarizing comments regarding mixed liquor solids control are offered:

1. Concentrations of 1,500 to 2,500 p.p.m. of suspended solids in the mixed liquor appear to constitute general practice, although values above or below this range are often encountered where plant limitations and unusual local conditions are involved.
2. There appears to be an increasing trend toward recognition of the volatile matter content of the mixed liquor solids as a control factor rather than the suspended solids. This appears logical since the volatile matter is actually the active constituent of the sludge. A minimum volatile solids concentration of 1,500 p.p.m. is recognized as a control threshold in two of the plants represented.
3. It is suggested that there be no hesitancy in varying the mixed liquor solids to meet seasonal variations in load and temperature.
4. The importance of keeping the mixed liquor solids concentration within any limitations imposed by the air supply is emphasized. Proper operation cannot be obtained when the solids content is so high that the available air supply is inadequate to maintain an aerobic environment.
5. While low concentrations of solids may result in economy of power (for air supply) the volatile content and activity will be high whereas high solids concentrations usually yield less active but denser sludges which are more conducive to stable operation under shock loads and afford a somewhat higher degree of purification.
6. The rate of activated sludge accumulation or build-up is usually greater when the mixed liquor solids concentration is in the low range; the resulting high volatile content is desirable from a fertilizer production standpoint but entails closer control in avoiding septicity during compaction in the final settling tank.
7. High mixed liquor solids concentrations are ordinarily advantageous from the sludge disposal standpoint, since part of the volatile matter is oxidized during the aeration process and the greater stability of the sludge permits it to be concentrated to a lower moisture content before waste to disposal facilities.

Rate of Sludge Return

The rate at which the activated sludge is returned to the aeration tanks generally determines the condition of the sludge. Too low a rate of return may result in septicity and consequent bulking and other evils; too high a return rate may not allow sufficient time for concentration in the final settling tank and thus results in a light, voluminous sludge, creating problems in disposal of the excess going to waste. Obviously, the rate of return is important as regards mixed liquor solids control and a balanced procedure must be achieved.

Rates of return (expressed as percentages of the sewage flow), with return sludge suspended solids data, are listed in Table II. Of the 21 plants listed, 8 employ return rates between 20 and 30 per cent, 10 employ rates between 30 and 45 per cent and only 3 return at rates under 20 per cent. The very low return rate of 3.5 per cent practiced at Rockville Centre is largely explained by the very high solids concentration of 26,700 p.p.m. in the return sludge. In winter the solids concentration becomes as high as 35,000 p.p.m.—a remarkable condition when the unusually high volatile content (88 per cent in mixed liquor) is considered.

Examination of the return sludge suspended solids data contained in Table II indicates that it is difficult to determine a "normal" range. Nine of the plants achieve return sludge concentrations of 4,000 to 8,000 p.p.m. and seven plants report average values between 10,000 and 15,000 p.p.m. As is to be expected, it will be noted that the concentration of return sludge

solids is generally, although not always, lower at the higher rates of return.

The APHA committee report includes an interesting analysis of return sludge practice in the data showing the "ratio of return solids to incoming solids at aeration tanks." Ratios between 10:1 and 39.6:1 are shown for eleven plants; however, seven of these plants reported ratios between 10:1 and 20:1. These figures may possibly be better visualized if regarded as "pounds of suspended matter returned as activated sludge per pound of suspended matter in the sewage received for treatment at the aeration tank."

In his regulation of the rate of sludge return, the operator is almost always guided by the depth of accumulation or sludge blanket in the final settling tanks. Too deep a sludge blanket may result in septicity of the sludge and upset of the biological balance; too shallow or no blanket may yield a thin, poorly concentrated return sludge. Seasonal changes in temperature and volatile content of the sludge often necessitate variation in the return rate.

At Fort Wayne, Brunner gives much credit to the air-lift bubbler piping (Fig 1) which affords a constant indication of the sludge depth in the final settling tanks at that plant. An excessively deep sludge blanket due to bulking or insufficient return is revealed immediately by the bubbler system and the return rate is increased accordingly. Automatic sludge level indicators utilizing photo-electric cells have been employed with varying degrees of success.

Lehmann at Hackensack considers the sludge blanket level as a most important control and makes hourly observations for the purpose of return sludge flow regulation. Particularly close supervision of the sludge blanket is also necessary at Pasadena, where generally high temperatures and high volatile matter content induce rapid septicity in the sludge accumulation if held too long in the final settling tank for thickening. Similar difficulty is experienced at Omaha, where packinghouse wastes and surges of septic sewage solids from the combined sewers after rains create a problem. Ahrens reports that increased rates of sludge return at such times maintain the sludge in better condition and are beneficial in freshening the mixed liquor. This not uncommon practice of returning sludge at very high rates to smooth out shock loads is suggestive of the recirculation theory as applied to modern trickling filter treatment—concentration of the return sludge being sacrificed for the dilution effect of the effluent in which the sludge is suspended. The thin return sludge, however, introduces other problems in connection with waste and disposal.

Smith at Lima employs an increased sludge return rate at the apparent onset of bulking of sludge, which common operation difficulty will be discussed in more detail later. A semi-automatic return sludge rate controller has been developed at the Lima plant, by adapting a weighted plug valve to the return sludge pump discharge piping. The arrangement is shown diagrammatically in Fig. 2 and its operation is described by Smith as follows:

"A 4-inch valve in the return sludge pump header continuously discharges excess activated sludge to the primary settling tank for removal from the secondary system. The amount of such excess sludge is affected immediately by a back-pressure plug (Fig. 1) in the discharge of return sludge to the mixed liquor channel and *generally* by the settling rate of aerated sludge in the final settling tanks through control of the suction lift of the return sludge pump. Specifically, operating directions call for the individual sludge discharge valves between the final settling tanks and the return sludge well to be throttled at all

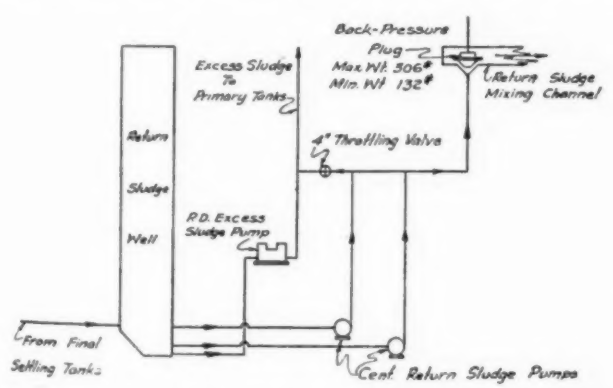

Fig. 2.—Diagram of return and excess sludge control arrangement at Lima, Ohio. E. E. Smith, Superintendent.

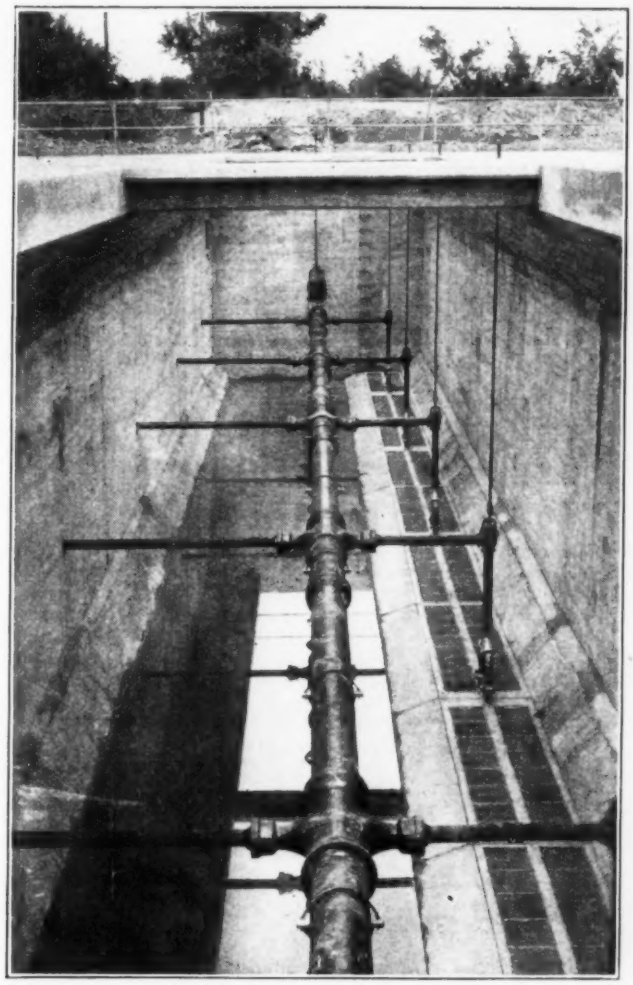

Fig. 3.—Empty aeration tank at Springfield, Ill., showing air piping and plate containers.

times sufficiently only to hold the sludge line in the final settling tanks at the edge of the vertical side-walls, maintaining the three foot cone in the final tanks full of sludge. If the rate of settling increases (or, conversely, the hourly percentage settling decreases) the more compact sludge will require less volume in the return and excess flows, the opposite conditions obtaining with a slower settling sludge.

"The back-pressure plug in the return sludge discharge, with empirically determined amount of weight will proportion the excess flow according to the rate of return and, indirectly, the sludge settling rate. Operating directions also call for some correction when the solids in the aeration effluent exceed a set figure. Thus, for the summer of 1941, when the solids in the aeration effluent exceeded 1,500 parts per million, the 4-inch return sludge bleeder valve was closed and the 50 g.p.m. excess sludge diaphragm pump operated for the three-hour period between 5:00 and 8:00 a. m. the day following that on which the determination of aeration effluent suspended solids was made. This three hour pumping seemed to be sufficiently in excess of the normal production of secondary solids so that no great deviation from the desired amount of solids followed such procedure.

"The arrangement may be called semi-automatic control of mixed liquor solids, as the system involves no effort on the part of the superintendent to make any detailed schedule of excess pumping as such. During the earlier part of the year when run-off provided a weaker combined sewage, the setting of 1,000 p.p.m. for aeration effluent was easily obtained by increasing the weights on the return sludge discharge plug. In addition to relieving the operators from the necessity of guessing at the number of hours of excess sludge pumping, this system has the following advantages:

"(1) Nearly eliminates the operation of the excess sludge pump, with its eight rubber diaphragms and four rubber ball valves, avoiding their maintenance.

"(2) Greatly reduces power for direct excess sludge pump motor operation, saving about $10.00 per month.

"(3) Has an apparent beneficial effect on primary settling, in that acting as a coagulant, the results from adding continuously small amounts of excess activated sludge are preferable to adding the same volume over shorter periods."

Rate of Air Application

Maintenance of aerobic conditions throughout the activated sludge process (aeration and final sedimentation) is essential to proper operation. Since the provision of an adequate air supply at suitable pressure almost always constitutes the principal operation cost item, the operator must exercise such control of air application that there is a sufficient quantity to afford stirring of the mixed liquor with a residual of dissolved oxygen in the mixed liquor during aeration, but to avoid the waste of power, undesirable changes in the activated sludge and over-nitrification of the effluent which occur when the air application is excessive.

The quantities of air applied in the plants represented in this review of operation practice are shown in Table III, and are expressed in cubic feet per gallon of sewage treated as well as cubic feet per pound of 5-day B.O.D. removed. The latter unit is rapidly gaining favor among operators as being most informative and more indicative of the manner of air utilization. Of the plants listed, nine show air requirements of 350 to 850 cu. ft. per pound B.O.D. removed and at only two, Omaha and Rockville Centre, is it found that more than 1,500 cu. ft. is required. The maximum application of 2,180 cu. ft. per pound B.O.D. removed at Rockville Centre may be explained by the intensely dense and active return sludge, which has been commented upon previously. In reviewing these data, it must be kept in mind that strict economy of air application is not so essential at plants equipped with sludge gas engine operated blowers such as at Findlay, Fort Wayne, Omaha, Rockville Centre, Springfield and others, in which circumstances liberal quantities of air may be used.

On the basis of the sewage volume treated, the average air application values in Table III range from 0.35 to 2.0 cu. ft. per gallon, with fairly general distribution between these limits.

TABLE III—APPLIED AIR QUANTITIES AND MIXED LIQUOR OXYGEN CONTENT

Plant	Applied Air (Cu. Ft.) Per M. G. Average	Range	Av. Per Lb. B.O.D. Removal	D.O. During Aeration (p.p.m.) Inlet	Midpoint	Outlet
Chicago (Calumet)	0.35	0.22-0.52	500	...	...	1.5-5.0
Elyria, Ohio	0.94	0.65-1.55	1290	...	...	...
Findlay, Ohio	0.70	0.30-1.00	840	0	1.0	3.0
Ft. Wayne, Ind.	0.99	0.49-1.65	835	0.8	2.0	5.2
Hackensack, N. J.	0.60	0.41-1.10	640	0-3.0	0.5-3.0	0.5-3.0
Indianapolis, Ind	1.78	0.83-4.80	1020	...	...	2.0+
Lancaster, Pa. (North)	0.70	0.50-0.90	830	1.8	...	5.2
Lima, Ohio	0.99	0.45-1.52	1415	2.2	5.0	4.9
Mansfield, Ohio	0.37	0.21-0.40	370	...	...	2.9
Muskegon Hts., Mich.	0.47	0.29-0.57	345	0	1.0	2.3
New York (Wards I.)	0.57	0.38-0.75	470	...	...	3.4
North Toronto, Ont.	1.00	0.70-1.30	620	...	...	...
Omaha, Neb.	1.36	1.02-1.86	1650	...	...	3.2
Pasadena, Calif.	1.62	1.47-1.89	1450	...	...	3.8
Pomona, Calif.	1.80	0.95-3.00	....	...	...	3.0
Pontiac, Mich.	1.10	.....	1470	...	...	5.0
Rockville Centre, N. Y.	2.00	1.70-2.30	2180	Tr.	1.5	2.5
San Antonio, Tex.	1.34	.....	970	...	...	...
Springfield, Ill.	0.75	0.43-1.10	950	2.0	4.0	6.0
Tenafly, N. J.	1.47	0.90-2.30	1295	...	...	...
York Twp., Toronto, Can.	1.70	1.60-1.90	....	4.6	2.7	2.7

The variation in applied air in each of the plants is not entirely due to manipulation by the operator because the unit "cubic feet per gallon" is subject to variations in the rate of sewage flow received for treatment. Comparison of the cubic feet per gallon values with the figures representing cubic feet per pound B.O.D. removed in the plants at Chicago Calumet, North Toronto, Omaha and San Antonio will indicate clearly how misleading the former basis of expressing air application may be.

The concentration of dissolved oxygen in periodically collected samples of mixed liquor is the most commonly employed control for determination of the adequacy of air application (Table III). The frequency of such mixed liquor sampling and dissolved oxygen determinations varies from 1 to 12 times daily, with most operators using a routine 4-hour schedule. From Table III it will be observed that many operators determine the mixed liquor dissolved oxygen content at the inlet and midpoint of aeration as routine control in addition to D.O. determination at the aeration tank outlet. Brunner at Fort Wayne supplements his 12-hour aeration tank effluent sampling routine with a dissolved oxygen survey through each aeration tank at least once a month.

The mixed liquor dissolved oxygen data in Table III illustrate the progressive increase in residual oxygen which is nearly always found to occur during aeration. Although it is not uncommon to find a complete absence of dissolved oxygen at the aeration tank inlet, it is generally held desirable for some residual to be present throughout the entire aeration period. Residual oxygen must certainly be present at the midpoint of the aeration period if normal results are to be obtained. The midpoint oxygen concentration is considered the primary control at Toronto.

It has been demonstrated experimentally and in practice that a normally active sludge uses, in the first two hours of aeration, about 50 per cent of the total amount of air it requires. This finding constitutes the basis for the practice of tapered aeration, whereby air application is heaviest at the inlet end of the aeration units and gradually reduced to a minimum at the outlet end. Most of the larger plants do not employ tapered aeration although the principle has many enthusiastic supporters.

In controlling air application by means of the dissolved oxygen content of the aerated mixed liquor, Lehmann at Hackensack seeks to maintain from 1.0 to 2.0 p.p.m. and makes adjustment on the basis of observations every two hours. Such unusually close control appears to be required by a sensitive activated sludge which affords a narrow margin between under and over-aeration. At Omaha, Ahrens considers 1.5 p.p.m. as the lowest desirable dissolved oxygen limit. A minimum of 2.0 p.p.m. appears to be the most commonly recognized control value and is employed at Fort Wayne, Indianapolis and other plants. Wheeler at Chicago (Calumet) encounters seasonal factors which lead to the use of a 1.5 p.p.m. control in summer and 2.0 p.p.m. in winter. At Wards Island Edwards finds it desirable to have about 5.0 p.p.m. at the aeration tank outlets at all times.

The dissolved oxygen content of the plant effluent is, of course, a most important item and is often recognized as the principal control, as at Lima, or is considered in conjunction with the mixed liquor oxygen concentration, as at Fort Wayne, Wards Island and Omaha. At Lima, Smith attempts to maintain at least 3.0 p.p.m. of dissolved oxygen in the plant effluent and finds that 3.5 to 4.0 p.p.m. must be held in the aerated mixed liquor to achieve the desired end result.

Brunner (Fort Wayne) and Turner (Mansfield) direct attention to the particular significance of mixed liquor dissolved oxygen determinations made at critical flow periods during the day. Turner is guided entirely in adjustment of air application by the decrease in dissolved oxygen which occurs during the peak load period of 10:30 a.m. to 2:00 p.m. If the dissolved oxygen drops as much as 75 per cent in this time, air application is increased; if the drop is as low as 50 per cent, less air is used. This procedure appears to have particular merit where shock loads are received.

Where there is poor flexibility of air supply, as at Elyria, or inadequate blower capacity, as at Muskegon Heights, the mixed liquor oxygen content can be controlled only by variation of the concentration of mixed liquor solids carried. Barton (Findlay) and Rhoads (Lancaster) find it desirable to maintain a constant rate of air application and to make adjustment only by increasing or decreasing the mixed liquor solids, even though no design limitations affecting the air supply are involved in these plants.

An interesting observation is offered by Lehmann regarding use of the mechanical paddles used in combination with diffused air at Hackensack. It is found that a decrease in mixed liquor dissolved oxygen occurs when the paddles are started, apparently demonstrating their effectiveness as a mixing device. The paddles are operated only during the summer months of heaviest load.

At Tenafly, New Jersey, Adams finds that more air must be applied than is required for biological balance of the activated sludge in order to maintain proper admixture and circulation of the mixed liquor.

Aeration Period

Although discussed here as an operating variable, the control of the aeration period by actually varying the detention time in aeration units is not common in practice, since few diffused air activated sludge plants have the excess aeration tank capacity necessary to make the procedure possible. It is feasible, however, for the operator to utilize the available aeration tank capacity in various ways, as by reaeration or by "adjusted aeration" (used at Findlay, Ohio), both of which procedures are discussed below.

The aeration periods available in the plants represented in this study are shown in Table I. At ten of the plants listed, periods between 4 and 6 hours are found; at five plants the aeration periods are within the 6 to 8 hour range, and periods in excess of 8 hours are reported at the remaining six plants. In view of

the wide variation in available plant capacity thus indicated, it is not surprising that there exists such a broad range in operating practice in regard to mixed liquor solids concentration, rate of sludge return and rate of air application. All of these more readily controlled variables must be adapted to suit the less flexible aeration period.

Longer aeration periods usually yield a higher degree of purification with more advanced nitrification. At some plants, at which algae growth in the outlet watercourse may present a problem, such higher nitrification may be a disadvantage rather than desirable. Short aeration periods ordinarily require more "high-pressured" operations, unless a weak sewage is received for treatment, and demand more care on the part of the operator in developing and maintaining a well-conditioned activated sludge. The plant having liberal aeration capacity is usually less subject to upset by shock loads, as at Lancaster, Pa., and Pontiac, Mich. At Pontiac the activated sludge plant is augmented by a separate trickling filter plant, hence a constant, controlled sewage flow is handled by the former works, greatly simplifying all phases of operation.

Lehmann at Hackensack suggests the desirablity of plant design details permitting flexibility in aeration capacity. Faulty hydraulics apparently make it impossible to remove aeration tanks from service as may be desired.

Return Sludge Reaeration.—Routine, continuous reaeration of the return activated sludge before admixture with the aeration influent sewage is practiced in only two of the plants contributing to this study, *i.e.*, San Antonio and Pasadena. At the latter plant a 45 to 80 minute reaeration period is utilized primarily as a means of return sludge distribution. Reaeration is employed at Muskegon Heights on occasion "to lighten a too heavy sludge and to relieve floating sludge conditions in some cases." Anderson (Muskegon Heights) refers to the highly volatile activated sludge, particularly since packinghouse wastes have been received, and difficulty in properly conditioning the sludge with an inadequate air supply as being the causes for resorting to reaeration. When the sludge approaches a septic condition, reaeration is accomplished by discharging a portion of the return sludge to the last aeration tank, at a point about 25 feet from the outlet end. Air application to this unit is correspondingly increased.

The following local conditions justify the practice of return sludge reaeration by Berg at San Antonio:

1. A strong sewage containing industrial wastes, subjecting the plant to abrupt increases in load as when sudden flushing of sewers occurs.
2. High prevailing temperatures which induce septicity in the sewage and activated sludge solids.
3. Budgetary restrictions on power purchases, resulting in some limitation of air supply.
4. Unusually wide variation in rates of sewage flow, resulting in extremely long final settling tank detention periods in the early morning hours.

A reaeration period of about 0.9 hour is employed at San Antonio, as compared to the mixed liquor aeration time of 5.9 hours. The reaerated sludge averages about 4,200 p.p.m. and settles to 88 per cent after 30 minutes as compared to 1,500 p.p.m. and 33 per cent, respectively, for the mixed liquor. Air application was at the rate of 880 cu. ft. per pound of 5-day B.O.D. in the aeration influent sewage although Berg concludes that 1,000 cu. ft. per pound applied B.O.D. is the minimum quantity which will maintain a well-conditioned sludge under San Antonio conditions.

There are two fundamental principles in general acceptance which should be recognized when return sludge reaeration is considered: first, that the air application facilities must be of adequate capacity to maintain aerobic conditions in the reaeration unit, and, second, that prolonged absence of food is likely to render the sludge inactive and low in oxidizing capacity.

"Adjusted Aeration" (Findlay, O.).—An interesting routine is employed by Barton at Findlay, O., during low flow periods to reduce the aeration time and to minimize the effect of surge flows which result in disturbance of the final tank sludge blanket with loss of sludge floc at the weirs. The method involves alternating the operation of the two aeration units at eight-hour intervals by the simple expedient of closing the valve on the discharge pipe to the final settling tank, thus permitting the unit thus shut off to "idle." The water level rises in the idling unit, creating an increased back pressure at the plates and diverting most of the air to the working unit. The resultant idling and working cycle actually effects a substantial measure of reaeration of the sludge and has apparently eliminated the surge influence at Findlay. In times of normal sewage flow, both aeration units are operated in parallel and in conventional fashion.

Condition of Activated Sludge

The condition of the activated sludge at any time reflects the end-point of all manipulations by the operator, hence is his most important guide. Were it not for the lack of constancy of the character and quality of the sludge, the operation of an activated sludge plant would offer no complications of consequence. The observations upon which most dependence is placed at this time in determining the condition of activated sludge are:

1. Appearance and odor.
2. Settling characteristics and density, observed by laboratory observation of the rate of settling and computation of the sludge index.
3. "Activity," as observed directly by measurements of the rate of oxygen utilization or as estimated indirectly from the proportion of volatile matter present.
4. Microscopical examination to ascertain the types and prevalence of the biological life present.

Other determinations of such factors as pH and dissolved oxygen afford information concerning the environment of the sludge.

Appearance and Odor.—When in good condition, activated sludge is ordinarily light to moderate brown in color, becoming dark brown to black as septicity may occur. The odor of well-conditioned sludge is sharply musty and not unpleasant but turns sour and disagreeable when in a very bad state. The flocs making up good quality sludge are of moderate size and have clearly defined edges; a poor sludge usually appears to take the form of spongy, fluffy masses. During times of extreme low temperatures, the flocs may become quite small and dispersed.

Settling Properties and Density.—The settling properties of activated sludge are the most significant indexes of the sludge condition since efficient operation is dependent on the ready separation of the sludge from the treated liquor in the final settling tank. Well-conditioned sludge normally occupies about 20 to 30 per cent of the mixed liquor sample volume after five minutes' settling in a graduated cylinder. The most common manifestation of poor sludge condition is "bulking" (to be discussed in detail below), in which state the sludge particles become light, enlarged and difficult to remove by sedimentation. The sludge index is an expression of the density of the sludge and is more informative than the settling test alone since it takes into account the amount of sludge solids contained in the mixed liquor sample. Sludge index values below 100 are generally considered representative of well-conditioned sludge although satisfactory operation may be accomplished with sludges having indexes in the order of 200, at Wards Island, Pasadena and San Antonio. It may be significant that the activated sludges in these plants are all of high volatile matter content. The effect of industrial wastes containing solids of high or low specific gravity should also be recognized in the interpretation of sludge index values and, particularly, in the determination of optimum operating ranges for a specific plant.

Sludge Activity.—The determination of sludge activity by measurements of oxygen utilization as developed by Bloodgood at Indianapolis has been adopted at a number of plants. By means of a machine developed for the purpose (see Sew. Wks. Jour., 10, 927, November, 1938), the amount of oxygen, in p.p.m. per hour, used by samples of mixed liquor from each aeration tank, is determined daily. This rate of oxygen utilization is termed the "sludge demand." The samples are taken from the outlet end of the aeration tank and concentrated to a standard value of 0.5 per cent solids for the measurement. The sludge from this test is then fed with synthetic sewage to produce a 0.5 per cent mixture and the rate of oxygen utilization again determined by the machine. The last determined rate minus the sludge demand yields an oxygen utilization rate, expressed in p.p.m. per hour, denoted by Bloodgood as "sludge activity." For best operating results at Indianapolis, the sludge demand must be kept in the range of 25 to 50 and the sludge activity should not exceed 70 to 100 p.p.m. per hour. The procedure is found useful in controlling air, mixed liquor solids, rate of sludge return and in ascertaining the shock overloads from industrial wastes. Further details will be found in Sew. Wks., Jour. 10, 26 (January, 1938).

Other devices, including the *Odeeometer*, have been used for determining rates of oxygen utilization by activated sludge and sewage-sludge mixtures.

In at least eight of the plants represented here, important significance is attached to the volatile matter content of the activated sludge as an operation control. Rhoads and Bolenius at Lancaster, however, find that there appears to be no uniform relationship between the volatile content and general characteristics of the sludge, but point out that this is probably due to industrial waste variations. At Chicago (Calumet) and Fort Wayne, the primary concern appears to be maintenance of the volatile suspended solids content of the mixed liquor above a minimum of 1,500 p.p.m., in order that a sufficiently active sludge will

obtain. At Wards Island, Mansfield, Muskegon Heights and other plants, operating difficulties and increased air requirements are attributed largely to high volatile solids contents of the return sludge. Turner at Mansfield reports that the volatile content of the return sludge solids averages about 80 per cent, which may explain the high prevailing sludge index (346). Increased quantities of slaughterhouse wastes received at Muskegon Heights raised the volatile suspended solids in the influent sewage from 65 per cent to 80-85 per cent, adding complications in operation control. The industry has been requested to install adequate grease removal and rate-of-flow control facilities in an effort to minimize the influence of the wastes.

Edwards, in comparing data from the New York Wards Island and Tallmans Island plants (Sew. Wks. Jour., 12, 1077, Nov., 1940), shows a marked relationship between the volatile solids content of the return sludge and the sludge index. The relatively high sludge index of 170 at Wards Island is held to be caused by the high volatile solids content of 76.1 per cent. Edwards suggests the sludge index may be a relative figure as an indication of bulking and that the variance in sludge indices encountered in comparing plant data may be explained by the differences in volatile matter content.

Edwards and Anderson (Muskegon Heights) both refer to the requirement of additional air application when the volatile content of the sludge is high, indicating that another important operation control relation may exist. There appears to be considerable justification for greater emphasis on the volatile solids determination in the mixed liquor and return sludge as an operation control, as well as for extensive study of the significance, interpretation and application of such data.

Microscopic Examinations.—Determination of the activated sludge condition by microscopic observance of the number and types of organisms present is practiced at most of the plants contributing to this study; however, the frequency of such examinations varies widely. At nine plants it is reported that the microscope is used "occasionally"; weekly sludge examinations are made at Lancaster and Pontiac; Edwards reports that five to six observations are made weekly at Wards Island. The occasional examinations made at Mansfield, Pontiac and Springfield are augmented by daily checks when the sludge index is high or when the sludge condition is otherwise doubtful. Bloodgood states that the microscopic observations are made seldom at Indianapolis "as it is believed that by the time there is a change in flora it is too late to make the necessary changes in operation procedure for maintenance of a good activated sludge."

At Wards Island, Edwards considers protozoa and filamentous forms as of most importance and finds flagellates to be numerous with low dissolved oxygen conditions. Larson at Springfield considers a preponderance of rotifera with ciliates such as *vorticella, carchesium, stentor,* and *paramoecium* to be present when the sludge is in good condition, an unsatisfactory sludge being characterized by the presence of *amoeba* and filamentous types including *Sphaerotilus natans.* Most common practice, typified by the procedures at Lancaster, Mansfield and Pontiac, is to check primarily for the filamentous growths, as these forms are usually prevalent in a poor sludge and are readily identified.

Other Measures of Sludge Condition.—Lehmann at Hackensack employs pH determinations on mixed liquor samples during aeration as an indication of the degree of oxidation of the sludge. He finds that a drop in pH occurs during normal operation and that bulking occurs otherwise. Too low a pH, of course, is evidence of septicity and must be avoided by higher return rates. At Omaha, Ahrens also includes routine pH determinations on the mixed liquor and return sludge among his observations for place at a pH of 6.8 in the return sludge and includes the sludge condition. Adams at Tenafly finds best operation to take place at a pH of 6.8 in the return sludge and includes the determination in his daily routine.

Determination of the ammonia, nitrite and nitrate nitrogen on the plant effluent are obviously informative regarding the oxidizing and nitrifying properties of the activated sludge and are commonly performed as routine analyses. From 1 to 2 p.p.m. of nitrates in the plant effluent is generally considered to indicate a sufficiently nitrifying sludge. Effluent nitrate contents of more than 5.0 p.p.m. are unusual since most operators consider it uneconomical to carry nitrification beyond this point. At Pasadena, where high temperatures entail constant vigilance against bulking, Allen considers the presence of at least 1 to 2 p.p.m. of nitrates in the effluent as an important safety factor. Anderson at Muskegon Heights has found that methylene blue stability determination affords a quick and simple check on nitrate production and includes this test on the plant effluent in his routine. Immediate loss of color is considered to indicate a complete absence of nitrate nitrogen. Anderson also suggests that the turbidity determination may give indication of upset in sludge condition even though results of other determinations may appear satisfactory.

Bulking of Activated Sludge.—Well-conditioned activated sludge settles rapidly from the aerated mixed liquor leaving a sparkling, highly oxidized plant effluent. Loss of the rapid settling property and increase in the sludge index is usually an early indication of deteriorating sludge quality. As the flocs increase in size and lose density, the sludge blanket in the final settling tank increases in depth until the natural currents in the tank carry some of the solids over the weirs with the plant effluent. Avoidance of such bulking of the sludge is the most common problem encountered in operation and activated sludge plants in which it has never occurred are indeed rare. Of the plants represented here, "chronic" bulking is reported at four; one or more instances of bulking each summer are experienced at eight; at nine of the plants bulking is reported to occur infrequently or only "occasionally in mild form." Included in the last group are the plants at Hackensack, where no bulking has occurred in the past two years, and at Rockville Centre, where there have been no such difficulties in the past eight years, improved operation control being credited for these records in both cases.

The reported causes of bulking are summarized as follows:

1. Loss of balance between operation variables due to carelessness or relaxed vigilance in control is offered as the primary cause in six plants. Too high or too low mixed liquor solids concentration and inadequate air application are mentioned as specific reasons.

2. Shock loads from industrial wastes are held responsible for bulking experienced at five plants. Toxic wastes as from paint manufacture and metal plating plants, wastes of high organic content as from packinghouses and distilleries, and wastes containing appreciable quantities of oil and grease are considered most likely to result in interference. At Muskegon Heights and Lancaster, the industrial waste influence has been minimized by requiring such industries to effect removal of objectionable components and provide for uniform rates of discharge to the sewer system.

3. At four plants, high volatile matter concentrations, occurring usually in dry summer months when the raw sewage solids are likely to be septic, are blamed for activated sludge bulking. Onset of bulking following a rain which flushes out septic solids from the combined sewers after a prolonged dry period is common. Similar effects are reported when a concentrated supernatant liquor from digestion tanks is returned to the raw sewage (Omaha and Hackensack) and when the primary sedimentation tanks fail to retain completely the waste activated sludge solids discharged thereto at high rates (Omaha and Springfield).

4. Consistently overloaded plants are almost always operated under a condition of chronic bulking, a situation obviously beyond control of the operator. Wartime population increases together with boron wastes from citrus fruit packing industries has brought this condition at Pomona, Cal. Commenting on bulking experience at Lima, Smith states, "When an activated sludge plant is operated at full load under summer conditions, as the Lima plant was in 1942 for over 50 per cent of each day, bulking will occur." Many plants not now troubled with bulking will undoubtedly experience the problem as design loads are approached and exceeded in the future, unless plant expansion is accomplished in pace with increases in domestic sewage and industrial waste flows.

Wheeler at Chicago (Calumet) considers the infrequency of bulking at this plant to be due to the relatively low volatile content of the sewage received. At Pasadena, Allen has been unable to associate any specific causes to the sludge bulking which occurs at irregular intervals; however, relatively high temperatures such as prevail in California are generally believed to accelerate bulking.

It will be evident that continuously careful and vigilant attention to the control of mixed liquor solids, return sludge rates and air application will prevent many instances of bulking. Industrial waste survey and control will minimize cases resulting from such sources. In other cases bulking may be prevented by improvement in supernatant liquor quality or by alternate methods of disposal. At Hackensack, elimination of a heavy supernatant liquor, together with a higher mixed liquor solids level (1,600 p.p.m.) and improved dissolved oxygen control during aeration, have successfully prevented bulking for the past two years. Where shock loads from industries are likely to be received, it is desirable to have such knowledge early so that measures may be taken to prevent bulking. The industrial waste "slugs" received at Decatur, Ill., and Lancaster, Pa., are of such nature that pH determinations on the raw sewage at intervals during the day are regular procedure. An abrupt change in pH signals that a "slug" has arrived and enables adjustment in air supply or return sludge rate to be made before damage to the activated sludge takes place. Barton at Findlay and Adams at Tenafly use the

methylene blue stability test on samples of the primary effluent for the same purpose, immediate loss of color being taken as indication that air application and sludge return should be increased to meet a shock load. This determination is routine at 8-hour intervals at Findlay.

In the earliest stages of bulking, operators appear to be in general agreement that additional air application, increase in rate of sludge return and, if possible, reduction in the applied load to the aeration tanks, are the proper operating adjustments. Ahrens at Omaha supplements these controls with the return of plant effluent to the sewage entering the primary tanks during the low night flows, thus reducing detention periods and freshening the aeration influent sewage.

At five of the plants included in this review, *i.e.*, Lancaster, Lima, Mansfield, Tenafly and Wards Island, chlorination of the return sludge has been found efficacious in destroying the filamentous organisms, such as *Sphaerotilus*, which usually infest bulking activated sludge, as well as in reducing the high initial oxygen requirements of the sludge. Return sludge chlorination is relied on as a routine remedy for bulking at the above and many other plants although some instances have been reported in which the treatment was not completely successful. For best results, it appears that the treatment should be instituted as soon as bulking becomes imminent with the chlorine applied at carefully controlled rates.

Probably the best information available on return sludge chlorination comes from Lima, Ohio, where Smith has employed the method since 1934, from which experience it has been concluded that it is the only uniformly successful remedy for bulking in this heavily loaded plant. Application of chlorine to the return sludge is begun when the aerated mixed liquor settling test shows the sludge to occupy more than 20 per cent of the sample volume after one hour settling. In arriving at the optimum chlorine dosages, Smith has recognized the concentration of solids in the return sludge as an important factor and expresses the chlorine dosages on this basis. Since 1936, chlorination of the return sludge has been required 3 to 12 times a year, the dosages averaging about 8.5 p.p.m. chlorine for each per cent suspended solids in the return sludge and ranging from 3.9 p.p.m. to 14.9 p.p.m. for each per cent solids. These dosages represent an average of 6.5 p.p.m. and a range of 3.0 to 11.7 p.p.m. of chlorine on the basis of the volume of return sludge flow. From the Lima experience Smith concludes as follows:

"1. Return sludge chlorination for correction of bulking has been practiced at Lima for nine years with success.

"2. Chlorine should be used for correction of bulking only, not for prevention.

"3. The amounts observed at Lima to be required for quick results are not less than 6 p.p.m. based on return sludge flow or less than 8 p.p.m. for each per cent of solids in return sludge.

"4. Nearly complete recovery from bulking may be expected at Lima within seven days after the beginning of chlorination of return sludge, and recovery has been effected on many occasions after as few as three days' chlorination.

"5. With effective reliance upon chlorination as a corrective of bulking, power requirements for air compression may be reduced to the minimum, and the combination of the above considerations should result in marked economy of operation."

When bulking is beyond control, *i.e.*, application of the above remedies fails to bring about restoration of a properly conditioned activated sludge, most operators agree that all or part of the bulky sludge must be wasted and a new sludge developed. At Indianapolis, waste sludge is readily disposed of by discharge to lagoons, hence all of the bad sludge is wasted. Where sludge wasting and disposal facilities may be limited, however, it is not always possible to effect complete wastage of the bulked sludge in a reasonable length of time and a part of the original mixed liquor solids may be retained. Ahrens at Omaha wastes the mixed liquor solids to about half the desired concentration and uses an increased rate of sludge return while the new sludge develops. Anderson (Muskegon Heights) retains about 300 p.p.m. of the bulked mixed liquor solids when redeveloping an activated sludge even though he is not restricted in waste sludge disposition facilities. At Findlay, O., Barton notes marked improvement in the condition of the mixed liquor solids in 5 to 10 hours after purging of the final tank contents through the plant is commenced.

Edwards (Wards Island), Larson (Springfield) and Turner (Mansfield) refer to the benefits derived from a rain while sludge is bulking. This remedial effect is generally ascribed to the reduction of the volatile matter concentration in the sludge, resulting in better density and lower initial demand for oxygen.

Application to the mixed liquor of lime, copper sulfate and pulverized clay has been attempted as a corrective for bulking activated sludge but has not proven uniformly successful up to this time.

Other Operation Problems

Rising Sludge at Final Settling Tanks.—At almost half of the plants represented here, difficulty has been experienced at some time or another with the rising of large masses of the sludge to the surface of the final settling tank. At Fort Wayne, Hackensack, Pasadena and Springfield, the condition has been noted to occur when nitrification is high as is common when the sludge is over-aerated. The most common opinion on the cause of this condition is that the nitrates are reduced in the sludge blanket as the dissolved oxygen is depleted, resulting in the ebullition of nitrogen bubbles which carry the sludge masses to the surface. Correction is usually accomplished quickly by decreasing the air application. The plant at Hackensack appears to be particularly susceptible to this condition and the mixed liquor dissolved oxygen content must be held below 2.0 p.p.m. to avoid it. Bloodgood eliminates the condition by incresaing the sewage load to the unit causing trouble, thus furnishing additional bacterial food. When increase in load is not feasible, he suggests that the mixed liquor solids be reduced.

Rising sludge masses may also occur with the onset of septicity in a final settling tank blanket of very dense sludge, as at Muskegon Heights, where reaeration of the return sludge is employed as the remedy. Smith at Lima increases the return rate and applies a fine spray of water at the tank surface to break up the sludge masses, on the rare occasions that the problem has been encountered.

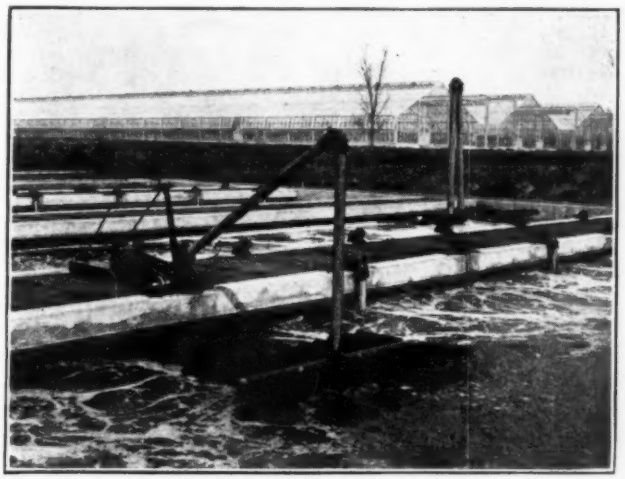

Fig. 5.—"Swing diffusers" raised for inspection and maintenance, Omaha, Neb.

Munroe at the York Township, Toronto, plant suggests that sludge rising due to septicity in hot weather may result from the failure of the sludge collectors to keep the tank bottom clean, directing attention to the desirability of care in maintaining proper adjustment of the plows or flights.

Oil and Grease.—In about a third of the plants contributing to this study, varying degrees of difficulty are reported to have resulted from the discharge of oils or grease to aeration units. Oils are usually received in "slugs" after accidental loss to the sewers or temporary breakdown of industrial separators. Grease is more likely to be a day-to-day problem since few industries provide adequate facilities and care in removing it from process wastes, particularly if the grease is in an emulsified state, difficult to separate.

Rapid and damaging reactions are reported to occur when a quantity of oil comes in contact with activated sludge, as experienced at Chicago Calumet, Fort Wayne, Hackensack and Omaha. In every case, air requirements were greatly increased and the clarifying and oxidizing capacity of the sludge immediately reduced, as evidenced by increases in the B.O.D. and suspended solids content of the plant effluent. Coating and impregnation of the activated sludge particles, interfering with the absorption of oxygen is believed to be the direct effect of the oil. At Fort Wayne, Brunner reports that a 3,000 gallon "shot" of fuel oil necessitated increase in air application from about 1.0 to 1.75 cu. ft. per gal. to hold 2.0 p.p.m. of mixed liquor dissolved oxygen. Receipt of 8,000 gallons of fuel oil at the Omaha plant completely destroyed the activated sludge, entailing a new start. Several discharges of a petroleum-base oil have been received at Hackensack and, although treatment efficiency is affected, the condition has been correctible by increased air application while the oil is present.

Grease may to some extent affect the activated sludge in a manner similar to oil but is believed to exert its greatest influence in the form of an organic overload. A combination of industrial oil wastes with grease solvent wastes constitutes a real problem at Muskegon Heights. At Elyria, Collier reports averages of 105, 33 and 3 p.p.m. of grease in the raw sewage primary effluent and final effluent, respectively, yet is more concerned with the grease disposal problem at the sludge digestion tanks than with the effect on the activated sludge. Adams at Tenafly states that determinations made several years ago reveal that the normal activated sludge at this plant contained about 20 per cent of ether-soluble grease but that attempts to correlate the grease content with the condition of the sludge were unsuccessful.

Chironomus Larvae.—In a few plants, a troublesome problem is created by the bloodworm or larvae of the *chironomus* fly. This insect has caused serious interference with operation of activated sludge units at Muskegon Heights and Findlay. Anderson (Muskegon Heights) furnishes an excellent commentary on the problem:

"The production of bloodworms in sewage is due primarily, I believe, to stale sewage and warm temperature. We have a considerable number of flat sewers and when the temperature of the sewage reaches about 70° F. the treatment plant usually becomes infested with bloodworms which have caused a great deal of difficulty with operation of the aeration plant.

"The worms form cocoons of sludge, about one inch in length by one-eighth inch diameter, which adhere to the walls of the tanks and finally break loose and float on the surface of the final settling tanks, due to entrained air or because the particles of sludge have become septic. The sludge may float in the original cocoon form or a very thin film of sludge may cover the entire tank surface. Also, small masses of sludge may rise to the surface causing very unsightly conditions and a loss of sludge. Invariably, during the period of infestation, the sludge is heavy, settles rapidly, and is in a semi-septic state.

"We finally reached the conclusion that the best remedy was to remove all of the sludge from the final settling tank sumps as rapidly as it was deposited. However, this required a return rate up to as high as 70 per cent and, when discharged to the entrance of the aeration unit, it shortened the aeration period and reduced efficiency. We then installed a sheet metal pipe from the return sludge chamber to a point about 25 feet from the discharge end of adjustable weirs so that the required portion of return activated sludge may be discharged to the first aeration tank with the remaining portion being discharged to the final aeration tank. We have four aeration tanks and larger amounts of air are applied at the first and last tanks during this method of operation. The method has prevented any sludge from floating on the final settling tanks during the past six years and apparently reduces the period of infestation to three to four weeks. Samples are collected for the laboratory and the rate of settling is watched closely, for when the period of infestation ceases the sludge index may rise rapidly and cause bulking. When the sludge index reached about 150 we return to the normal method of operating the aeration unit."

At Findlay, Barton describes the effect of *chironomidal* infestation as "devastating." Copper sulfate has been found efficacious in preventing the adult fly from laying eggs but is valueless after the eggs are deposited. After the larvae are present, practice at Findlay is to waste all of the infested sludge and start anew.

Maintenance of Air Diffusers

Among the most troublesome problems of physical maintenance of activated sludge plants is that of clogging and restoration of air diffusers. Diffuser clogging has become chronic in some plants after only a few months of operation; at others, no such difficulties have arisen in years. Among the latter are Pomona, Cal., where air diffuser tubes were used without being cleaned during a fifteen-year period; Springfield, Ill. (Fig. 3), where no clogging of plates has been experienced in almost fourteen years; and the plants at Findlay and Muskegon Heights which have been operating five years or more without experiencing diffuser clogging. Probably typical of average experience is that at Fort Wayne, which plant has been operating about two and a half years and where it is now necessary to replace the plates in one aeration tank and to institute a general schedule and procedure for restoring clogged plates. At Pasadena, Allen finds that annual cleaning of the plates in the older ridge-and-furrow tanks is required, usually after heavy rainy seasons, while some of the plates in the new spiral-flow tanks have not required cleaning in six years.

At some plants in which clogging is not a serious problem, a regular plate cleaning schedule is followed as a preventative. Harris at North Toronto drains the aeration tanks for plate cleaning operations once each year even though no appreciable increase in applied air pressure occurs in that interval. Diffuser plates at Omaha (in mixed liquor channels) and at Lancaster are occasionally "flushed" by the application of large quantities of air, this being the only attention required as yet in these relatively new plants.

Cause of Diffuser Clogging.—Although the direct cause of the diffuser clogging may not always be apparent, the following reported causes are typical of general experience:

1. Industrial wastes containing ferrous iron in sufficient concentration to form a hydrate as they are oxidized during aeration, which hydrate deposits on the top of the plate and penetrates into the surface pores. Clogging from this cause has been minimized at Chicago Calumet by diversion of pickling liquors from the sewers; such wastes in an average concentration of 65 p.p.m. total Fe in the settled sewage at Elyria create a serious clogging problem.
2. Organic growths which accumulate on the plate surface and restrict the pore openings, experienced at Indianapolis, Wards Island and Tenafly (only in old tanks in which the plates are of low permeability no trouble at more porous plates in new units).
3. Grease accumulations on top of plates, reported at Pasadena and Toronto.
4. Dirt and soot deposited on the underside of the plates by the applied air. Reported at Fort Wayne and Indianapolis. Recent examination at Fort Wayne also revealed an accumulation of paper fibers from the paper-cloth air filters.
5. Sand and silt impregnations at the top surface of the plates has caused trouble at Pasadena, Fort Wayne and Toronto.
6. Hard water scale is held responsible for the diffuser clogging encountered at Lima, Ohio.

Clogging of plates by activated sludge solids which subside to the bottom when the air supply may be interrupted by power failure or mechanical breakdown, is mentioned by several operators. When an aeration unit is to be removed from service for any reason, the application of air should be continued until the tank contents are drained to a level below the diffusers. Turner (Mansfield) and Adams (Tenafly) note that the diffuser plates clog more quickly when small quantities of air are being applied. Allen at Pasadena observes that the accumulation of grease deposits and silt at the plates increases rapidly when the plates become partially clogged.

Methods of Cleaning.—Diffuser cleaning procedures reported are summarized herewith:

Chicago Calumet—Ferrous iron deposits. Plates cleaned in place 2 to 3 times yearly. Washing and brushing followed by application of solution of equal parts water and sulfuric acid, to which sodium dichromate is added. Acid left in contact several hours, sometimes overnight. Second application given if clogging severe.

Elyria—Ferrous iron deposits. Plates cleaned in place annually. Deposits removed by application of hot flame which "spalls off" plate surface. Blow-torch used on silica plates, oxy-acetylene flame on carborundum.

Indianapolis—Organic growths, soot, dirt. Removable plates cleaned every two years. Plates are boiled in 6 per cent caustic solution for 5 hours, washed thoroughly and submerged in 10 per cent muriatic acid solution for 18 hours. No satisfactory method for cleaning plates grouted in place found as yet.

Wards Island—Organic growths and deposits. Plates cleaned in place every 1 to 2 years. Tanks drained, plates washed with hose, swept with deck brushes and allowed to dry (Fig. 4). Solution of 25 per cent caustic soda applied and left in contact for 24 hours. Caustic treatment repeated on plates in first pass. Plates put into operation without rinsing out caustic.

Hackensack—Organic deposits. Plates removed for cleaning annually. Soaked in 20 per cent caustic soda solution, rinsed freely and drained. Dipped into strong nitric acid solution and rinsed.

Mansfield—Organic deposits. Plates removed for cleaning every 2 years. Boiling and strong caustic solution followed by rinsing gives 96 per cent recovery of permeability.

North Toronto—Grease accumulations. Plates cleaned in place annually. Four holders at a time connected to low-pressure steam line and plate surfaces brushed vigorously with stiff-bristle brush. Thin, greasy scum removed thus.

Toronto—Plates cleaned in place twice yearly. Water jet at 110 pounds pressure with vigorous sweeping with street brooms, while plates subjected to normal air pressure, removes sand and silt. Grease accumulations removed by low pressure steam applications as at North Toronto.

Pasadena—Grease and silt. Plates in old ridge-and-furrow units cleaned annually. First washed in place with high pressure nozzle, followed by application of acid and rinsing. Treatment usually successful but if not, plates are replaced.

Rockville Center—Diffuser tubes cleaned in place weekly. Air valve on pipe to each diffusion unit closed and temporary hose connection made to city water supply. Flushing by water in this fashion has maintained tubes in good condition for eleven years. (Probably involves cross-connection to water supply, however.)

Lima—Hard water scale. Plates treated annually in place by application of weak muriatic acid solution.

Omaha—Chicago swing diffusers. Raising of diffuser units (Fig. 5) to enable them to be washed off with hose once a month, has sufficed during the first 18 months of operation of plant. Air pressure increase of about 0.5 lb. per sq. in. noted between cleanings.

When replacement of diffuser plates becomes necessary Allen (Pasadena) recommends that all plates in the same row or holder be changed at the same time. Use of a new plate in a holder containing old ones has been observed to disrupt the uniformity of air distribution because of the higher permeability of the new plate.

Waste Activated Sludge Disposal

Because of the high moisture content of waste activated sludge, its disposition almost always involves some difficulties which often influence control of the entire plant. Some problem associated with waste sludge disposal is reported at all but five of the plants represented here.

The most common method of effecting concentration of the waste activated sludge is by discharge to the raw sewage so that the waste solids are removed with the raw solids at the primary settling tanks. That this procedure is not without shortcomings is indicated by the fact that of twelve plants employing the method, only three fail to report some difficulty. Concentration of the waste activated sludge at the primary tanks at San Antonio has been found definitely unsatisfactory, probably due to the high temperatures which induce rapid septicity, and the high activated sludge index which prevails in that plant. At Fort Wayne, Muskegon Heights and Springfield, primary sedimentation efficiency is reduced when the waste sludge is light or bulking and some of the waste sludge solids pass on to the aeration tanks, imposing a recirculating load. Since increased waste is usually desirable under bulking conditions, inability to remove and concentrate the waste sludge is a serious limitation in the restoration of proper conditions at the activated sludge units. Lehmann (Hackensack), Brunner (Fort Wayne) and Larson (Springfield) emphasize the importance of frequent removal of the waste activated-raw sludge mixture from the primary tanks to avoid rising of sludge masses from the sludge hoppers. The air-lift bubbler device (Fig. 1) used at Fort Wayne for observation of sludge blanket elevations in the final settling tanks, has also been installed at the primary tanks so that excessive and overlong sludge accumulation can be avoided. At Lancaster, Mansfield and North Toronto, the waste activated-raw sludge mixture is high in moisture content, resulting in large quantities of digester supernatant as well as thin digested sludges. Alum is used to advantage in facilitating air drying of the thin digested sludge produced at North Toronto. Smith attributes the generally satisfactory removal and concentration of waste activated sludge in the primary tanks at Lima to the semi-automatic arrangement employed to control the rates of waste and return flow. To summarize this experience, it would appear that concentration of waste activated sludge at the primary tanks is reasonably satisfactory when the waste sludge is of low index, and that waste at low, uniform rates with frequent sludge removals from the primary tanks are highly desirable.

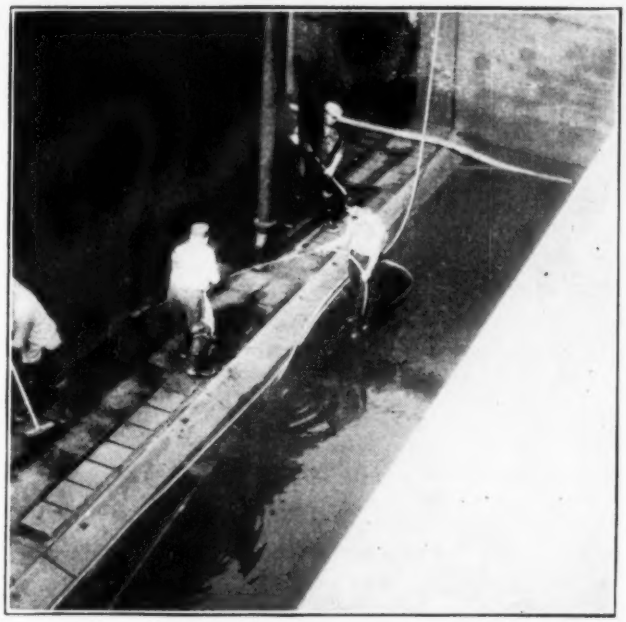

Fig. 5.—Cleaning diffuser plates at Wards Island plant, New York City

Separate decantation or concentration units are used at Rockville Centre before mixing the thickened waste sludge with raw sludge prior to digestion. Andersen reports no difficulties. The waste sludge is chlorinated at Wards Island to expedite concentration in the four final settling tanks used expressly for this purpose; concentration to 3 per cent solids is difficult, however. The concentrated sludge at Wards Island is pumped into tankers and disposed of at sea about 35 miles from the plant. Prompted by the need for sludge concentration at Findlay, Barton has conducted laboratory experiments on the thickening of the raw-waste activated sludge mixtures after dilution with digester supernatant liquor. One interesting but unexplained finding is that a 50 per cent water separation after settling occurs with a ratio of equal parts of supernatant and the mixed sludges, whereas a 40 per cent water separation is achieved when one part of supernatant is added to two parts of the sludge mixture. Plant scale facilities are not available at Findlay for further studies.

At Chicago Calumet, Tenafly and Toronto, the waste sludge is conditioned with chemicals and dewatered on vacuum filters. Adams at Tenafly finds that the character of the waste sludge exerts an appreciable influence on the dewatering operation; a light sludge giving poor filter yields while a heavy sludge will not cling properly at the bottom of the filter drum. The sludge index of the mixed liquor is adjusted to remedy these conditions.

The primary and waste activated sludges at Indianapolis are discharged to open lagoons for digestion and drying. The only problem as yet encountered is in bringing about removal of the dried sludge from the plant. About 25,000 to 35,000 cubic yards per year are sold and hauled away by local users but this is not quite up to the annual production.

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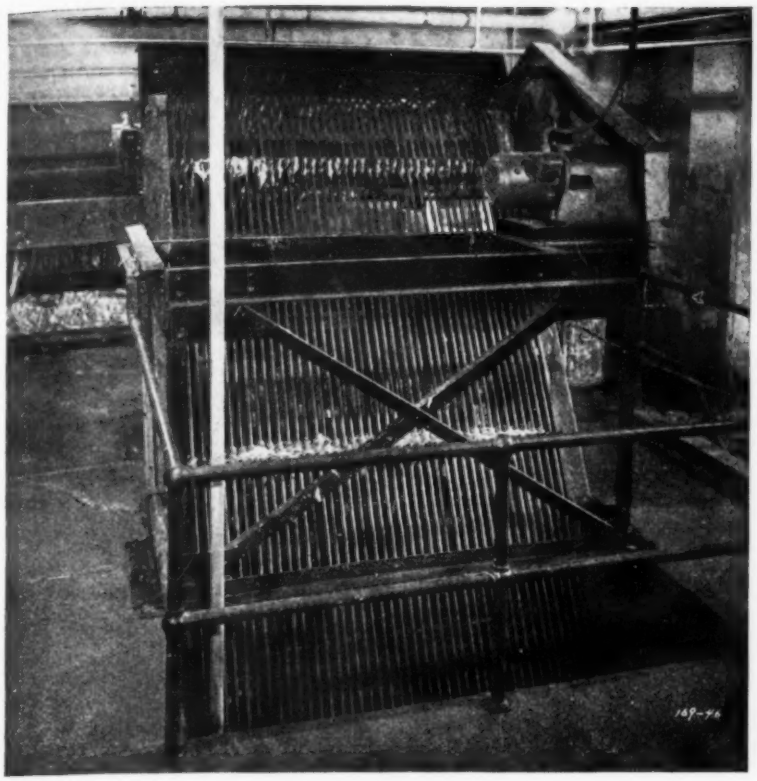

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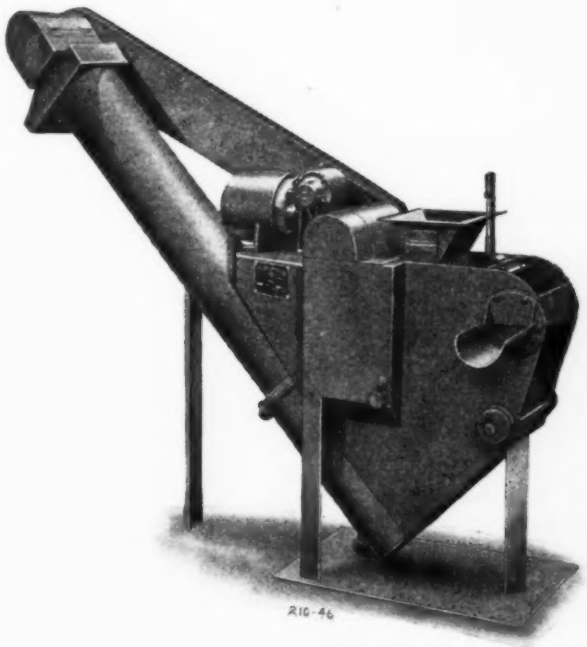

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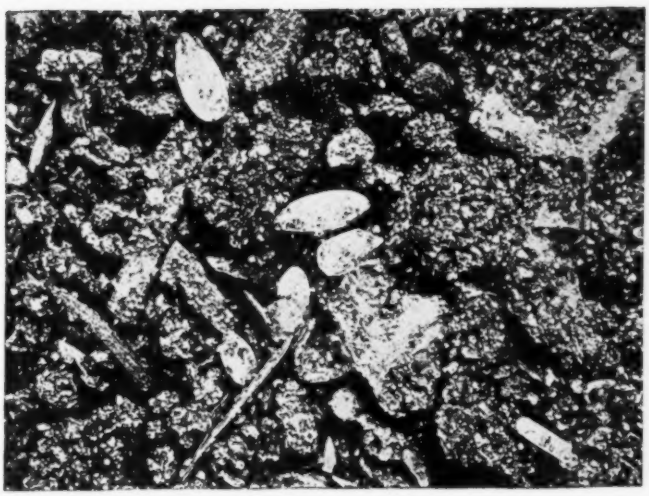

Magnified view of unwashed sewage grit

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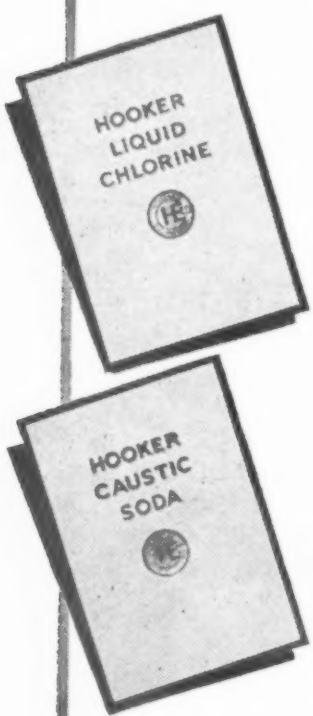
Along with Hooker's production of Chlorine, Caustic Soda and other chemicals goes the offer of help on any of your problems involving the handling and use of these chemicals. Hooker's Technical staff is continuously working with these chemicals and their specialized knowledge is available to help you overcome your difficulties.



HOOKER ELECTROCHEMICAL COMPANY

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Hooker Caustic Soda Hooker Caustic Soda is produced in our well-known and now nationally used Hooker type "S" electrolytic cells. It is of the same high purity and uniformity as other Hooker Chemicals. The flake form is shipped in drums of 100 or 400 lbs., the solid forms in drums of 220 or 700 lbs., and the liquid 50 and 70% solutions are shipped in drums of 700 lbs. or tank cars.

Send for These Booklets Hooker Chlorine and Caustic Soda Booklets contain helpful information on these two chemicals. Physical and chemical specifications, precautions in handling equipment for storing and shipping, uses and tests are included. Copies sent free of charge when requested on your letterhead.

Also available are copies of the Chlorine Manual, (published by the Chlorine Institute) which contains useful information about chlorine, its handling, employee protection, physical and chemical data and literature references.

HOOKER CHEMICALS

8293

PURGING AIR DIFFUSER PLATES WITH CHLORINE

By W. M. FRANKLIN

Supt. Water and Sewage Works, Charlotte, N. C.

The five aeration units at the Irwin Creek Sewage Treatment Plant, at Charlotte, N. C., are of the ridge and furrow type, equipped with diffuser plates. Manometers register the cubic feet of air per minute being delivered to each aeration unit. A master air-meter records the total quantity of air applied.

In the early summer of 1938 the diffuser plates in all the units became so badly clogged that it was impossible, even with two blowers in service, to get enough air through them to maintain proper aeration. All units were then cleaned out, as rapidly as they could be spared from service, and the diffuser plates scrubbed with muriatic acid. This had been the practice since the beginning of plant operation. All but two of the units responded reasonably well to this treatment. Units, No. 1 and No. 5, became clogged again after a few weeks operation. The acid cleaning was repeated in these two but with no better success. The normal average delivery of air per unit with one blower in operation is about 700 to 750 cubic feet per minute. The delivery of the diffusers in No. 5 unit was down to an average of 450 ft. with one blower in service, and with two blowers in parallel an average of 750 ft. could be maintained—(see Fig. 1). Since it was impossible to increase the air delivery in these two units by throttling the other three, and thus increasing the pressure of the air in the header feeding the diffusers, we were in something of a "mess."

At this time the plant load was heavy and the removal of one unit from service for a period long enough to empty and clean was extremely hazardous.

We decided to follow a suggestion previously made by L. H. Enslow of the Chlorine Institute. This suggestion was to add chlorine (as gas) to the air headers leading to the diffusers, with the idea that the chlorine would destroy algae growth in the plates, and might attack other clogging materials.

We first viewed this plan with alarm, fearing that the presence of chlorine gas in the air lines might cause corrosion, especially in the small feeder lines. Finally it was decided to try chlorine anyway. A tap was made in the header feeding No. 5 unit and chlorine fed into the air flow at the rate of about 17 lbs. per day.

In Fig. 1 the delivery of the diffusers in No. 5 aeration unit, in cubic feet of air per minute, has been plotted from August 22, 1938, through Oct. 9, 1938. Dotted lines indicate the period during which two blowers were operating in parallel, the solid lines indicating the period during which only one blower was in operation. Note that on Sept. 15 chlorine was fed for about four hours at the 17 lb. per day rate with two blowers operating. After the chlorine was cut off the two blowers remained in service the 18th. At this time one blower was removed from service and on the 19th the chlorine application was continued until the 150 lb. cylinder was exhausted on Sept. 25. The second blower was put on the line on the 24th with the idea that the increased velocity through the plates might help in opening up the pores. On the 30th normal operation was resumed with one blower.

Note the increase in air delivery during the period following the first "shot" of chlorine until the end of the curve on Oct. 9.

Actually, the chlorine brought about an increase of approximately 66% in air delivery with one blower in service and 53% with two blowers in service, as is shown by the curve. Needless to say we were so well pleased with the results that we immediately cleaned No. 1 unit with one cylinder of chlorine getting equally as gratifying results as was the case in the No. 5 unit.

At this date March 1, 1939, the air delivery of the diffusers in both Unit No. 5 and Unit No. 1 is about 750 cubic feet per minute and no detrimental effects from the application of chlorine have been observed.

It is our theory that the action of the chlorine not only cleaned the surface of the diffuser plate but also cleaned the pores *from the inside out,* removing organic matter from interior pores of the plate which could only otherwise be removed by a heat treatment, or long soaking in an acid bath.

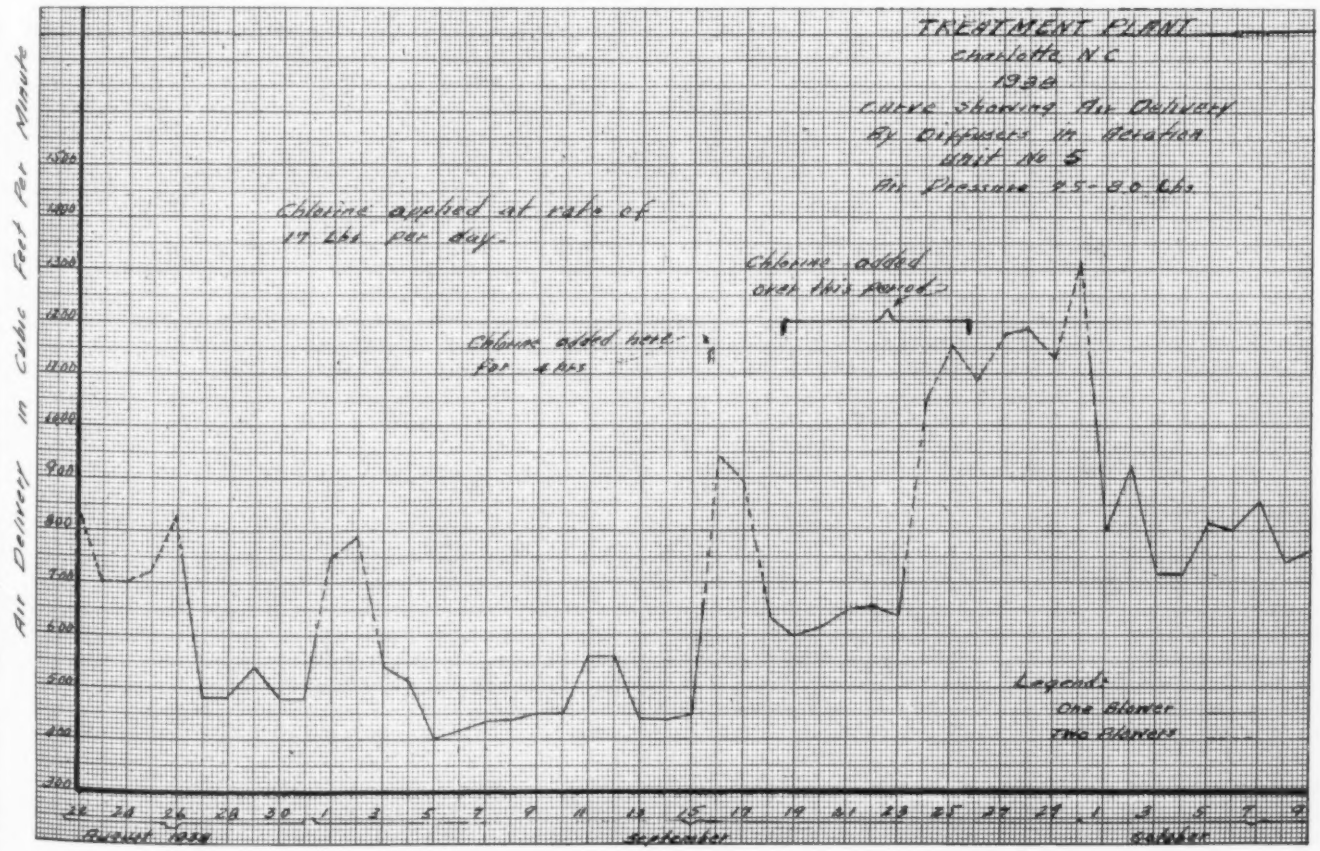

Fig. 1.—Graph Showing Change in Air Delivery Before and After Treatment of Plates With Chlorine Gas, in Situ

APPLICATION OF THE SLUDGE VOLUME INDEX TEST TO PLANT OPERATION

By DON E. BLOODGOOD

Prof. of Sanitary Engineering, Purdue University

THE Sludge Volume Index (S.V.I.) as given in "Standard Methods for Examination of Water and Sewage" has been a much discussed test. The original idea of the test was to indicate the bulking of activated sludge. The units used increase with an increase in the volume occupied by the same weight of solids. As a qualitative test it has been very helpful and a desire has often been expressed to correlate the sludge index with plant operation data.

If it is assumed that the concentration of sludge, obtained in the bottom of a cylinder, in a sludge index test, is the maximum concentration that can be obtained in a reasonable settling period, without detriment to the activated sludge, it is believed that the results can be used in a mathematical calculation which is applicable to plant operation.

It is realized, however, that the S.V.I. data obtained in a 30-minute settling period may not be the same as might be found for a 60-minute period (Fig. 1). It has been found from experience that even with mixed liquor which settles exceptionally well the concentration of sludge in the bottom of the test cylinder may be slightly higher at the end of 60 minutes than it is at the end of 30 minutes. The indications are that for mixed liquors of poor settling qualities the difference between the concentration at 30 minutes and 60 minutes is even greater. It would seem, therefore, that if the sludge index (S.V.I.) were calculated on a 60-minute settling period the results would be more satisfactory when used in the following calculations.

If the settling cylinder is considered as a secondary settling tank and the settled sludge as return sludge, the sludge index (S.V.I.) may be expressed in per cent solids of the sludge. Then per cent solids in the bottom of the cylinder may be obtained by dividing 100 by the sludge index (S.V.I.). This relation may be seen in Fig. 2.

This graph would be of assistance in activated sludge plant operation if the only desire were to determine the maximum return sludge concentration possible as indicated by the sludge index determination. The matter of sludge solids balance, however, enters the considerations and it is necessary to take it into account in discussing plant operation problems.

Development of Formulae

In an analysis of the sludge balance, for any one day of activated sludge plant operation, it is quite evident that the amount of solids in the mixed liquor must be equal to the solids in the return sludge, assuming the waste sludge, incoming raw sewage solids and the loss of solids by oxidation are considered negligible quantities. The mixed liquor return sludge balance may be expressed by the formula:

$$(a + b)x = b\,(y) \qquad (1)$$

a = sewage flow mgd.
b = return sludge flow mgd.
x = mixed liquor solids in %
y = return sludge solids in % (or solids in the bottom of settling cylinder)

If the sludge volume index be represented by (z), then, as above explained,

$$y = \frac{100}{z} \qquad (2)$$

The per cent by volume of return sludge may be expressed by the formula:

$$m = \frac{b}{a} \times 100 \qquad (3)$$

By simple algebraic substitutions of Formulae (2) and (3) in Formula (1) it may be written:

$$m = \frac{100}{\frac{100}{zx} - 1} \quad or$$

$$z = \frac{m}{(1 + .01\,m)x}$$

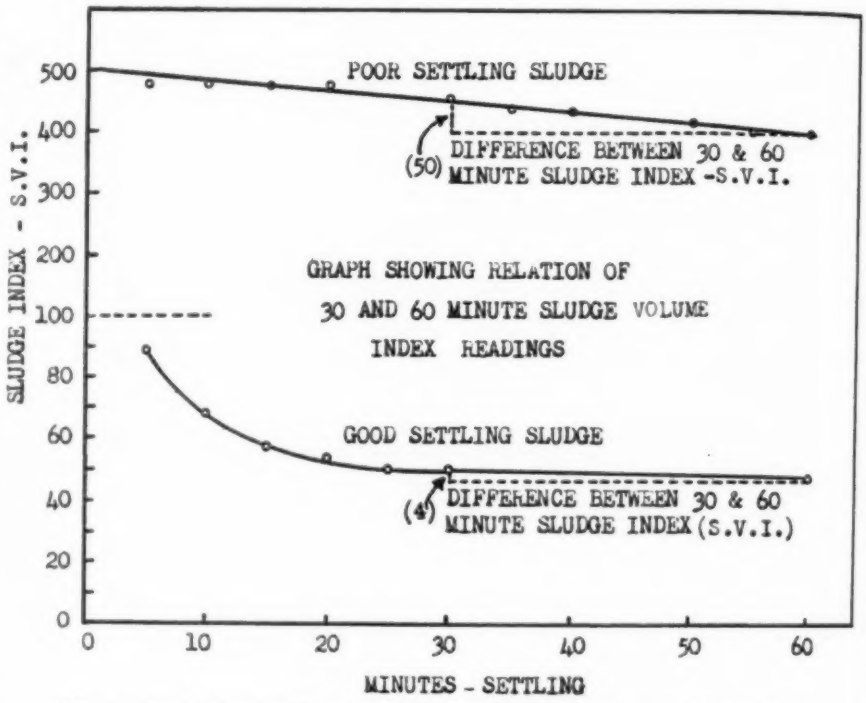

Fig. 1. Graph Showing Relation of 30 and 60 Minute Sludge Index Readings.

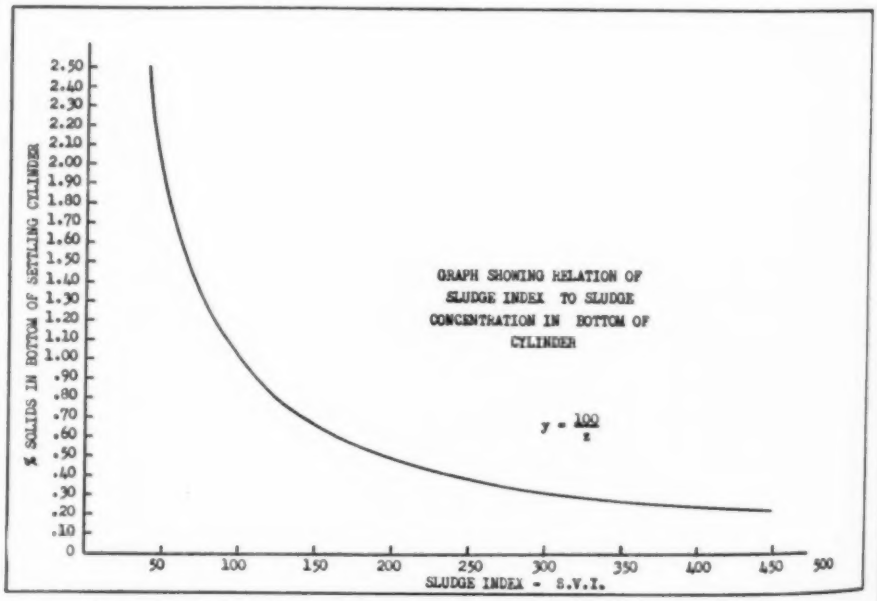

Fig. 2. Graph Showing Relation of Sludge Index to Sludge Concentration in Bottom of Cylinder.

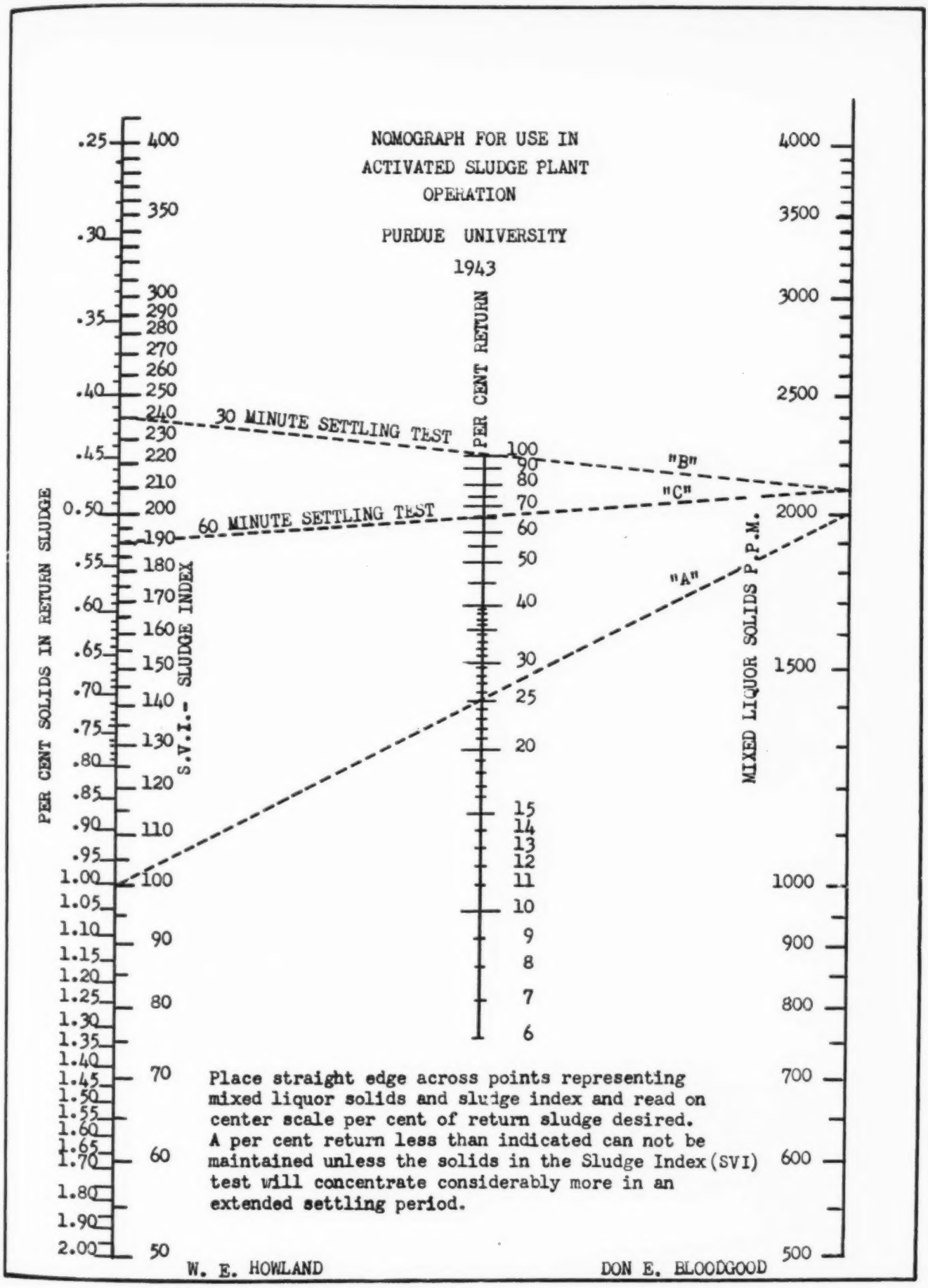
NOMOGRAPH FOR USE IN
ACTIVATED SLUDGE PLANT
OPERATION
PURDUE UNIVERSITY
1943
PER CENT SOLIDS IN RETURN SLUDGE
S.V.I.- SLUDGE INDEX
PER CENT RETURN
MIXED LIQUOR SOLIDS P.P.M.
30 MINUTE SETTLING TEST
60 MINUTE SETTLING TEST
"A"
"B"
"C"
Place straight edge across points representing mixed liquor solids and sludge index and read on center scale per cent of return sludge desired. A per cent return less than indicated can not be maintained unless the solids in the Sludge Index (SVI) test will concentrate considerably more in an extended settling period.
W. E. HOWLAND
DON E. BLOODGOOD

Fig. 3. Nomograph for Use in Activated Sludge Plant Operation.

A Useful Nomograph

Based on calculations with Formulae (2) and (3), a nomograph has been prepared (Fig. 3) which may be of assistance in activated sludge plant operation. Assuming the conditions as indicated by line "A," Fig. 3, with a mixed liquor solids concentration of 2000 ppm. and a sludge index of 100, it would be necessary to return 25 per cent by volume of return sludge of a concentration of 1.00 per cent. A higher per cent return could, of course, be maintained at an added return sludge pumping cost. On the other hand an attempt to return less than 25 per cent would mean holding the return sludge in the secondary tanks longer than seems advisable.

The assumed conditions as indicated by line "B," Fig. 3, with mixed liquor solids of 2100 ppm. and a sludge index of 240 (calculated on the regular 30-minute settling period) indicates the necessity of returning 100 per cent. Any amount of return less than 100 per cent would mean a rapid filling of the secondary settlers with sludge, unless the calculated sludge volume index at the end of a one hour settling test should be materially less than for a 30-min. test as illustrated by the line "C" in Fig. 3. In this case if a rate of sludge return of 65 per cent were maintained, the solids balance would be correct:

Comments:

1. If it is agreed that activated sludge should not be held in the secondary settling tanks longer than one hour it might be well to extend the period of settling of the sludge index test to one hour instead of the present thirty minutes.
2. The per cent of return sludge necessary for proper plant operation depends upon the mixed liquor solid concentration and upon the sludge index.
3. The nomograph presented is helpful in applying the sludge volume index test to the control of activated sludge plant operation.

The writer is indebted to Dr. W. E. Howland for his assistance in the construction of the nomograph presented.

E-Zee Bee-O-Dee

(*From the* DIGESTER *of the Ill. Dept. of Health*)

A SHORT, simple formula for the calculation of biochemical oxygen demand, developed by Professor H. M. Gifft of Cornell University, and published in the August, 1943, issue of *Water Works and Sewerage,* has been modified slightly by us [The Digester] for easier calculation.

The formula can be used for all types, strengths, and conditions of sewages, wastes, and polluted waters, with results identical to other accepted methods. It has the advantages of correcting automatically for the dilution water B.O.D. and of requiring only one dissolved oxygen test prior to incubation. In some cases even the D. O. Test may be estimated with sufficient accuracy. So—o—o, if you'd like to use something that makes your B.O.D work simple, try this method—it's easy.

Formula: $B.O.D. = (W - D) \times \frac{100}{p} + S - W$

W = D.O. in blank after incubation
D = D.O. in dilution after incubation
S = D.O. in sample before dilution
P = per cent of sample in dilution

Example 1: B.O.D OF RAW SEWAGE

Given: D.O. in blank after incubation = 8.0 = W
D.O. in dilution after incubation = 3.3 = D
D.O. in sample before dilution = 0.0 = S
per cent of sample in dilution = 2. = P

Then: $B.O.D. = (W - D) \times \frac{100}{p} + S - W$

Step 1: $(8.0 - 3.3) \times \frac{100}{2} + 0.0 - 8.0$

Step 2: $4.7 \times 50 + 0.0 - 8.0$

Step 3: $235 + 0.0 - 8.0 = 227$ ppm B.O.D (Ans.)

Example 2: B.O.D. OF PRIMARY EFFLUENT

Given: D.O. in blank after incubation = 8.4 = W
D.O. in dilution after incubation = 2.2 = D
D.O. in sample before dilution = 1.1 = S
per cent of sample in dilution = 5. = P

Then: $B.O.D. = (W - D) \times \frac{100}{p} + S - W$

Step 1: $(8.4 - 2.2) \times \frac{100}{5} + 1.1 - 8.4$

Step 2: $6.2 \times 20 + 1.1 - 8.4$

Step 3: $124 + 1.1 - 8.4 = 116.7 =$ ppm B.O.D. (Ans.)

Example 3: B.O.D. OF FINAL EFFLUENT

Given: D.O. in blank after incubation = 8.6 = W
D.O. in dilution after incubation = 2.6 = D
D.O. in sample before dilution = 6.4 = S
per cent of sample in dilution = 25. = P

Then $B.O.D. = (W - D) \times \frac{100}{p} + S - W$

Step 1: $(8.6 - 2.6) \times \frac{100}{25} + 6.4 - 8.6$

Step 2: $6.0 \times 4 + 6.4 - 8.6$

Step 3: $24 + 6.4 - 8.6 = 21.8 =$ ppm B.O.D. (Ans.)

EXPERIENCE IN MECHANICAL ACTIVATED SLUDGE PLANT OPERATION*

By DOUGLAS E. DREIER
San. Engr., Walker Process Equip. Co., Aurora, Ill.

CONTINUING the program of "The Operator's Corner" in presenting studies of actual operation procedures in fundamental sewage treatment processes, this article reviews control practices in mechanical activated sludge plants. It is largely based on data and comments obtained from 27 superintendents, engineers, chemists or operators of 29 representative plants through detailed questionnaires and through supplementary correspondence.

An effort was made to obtain an even distribution of returns from plants utilizing the three most common types of mechanical aerators and, while complete success in that regard was not achieved, it is believed that adequate distribution was obtained. It is not the purpose of this review to discuss and compare the relative merits of mechanical aeration equipment of the several types. Grateful acknowledgment is given the complete co-operation of the following contributors:

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A wide variation in capacity, loading and design is indicated in the general data for the plants contributing, as tabulated in Table I. The loading data tabulated in the ninth column of the table were calculated from the information submitted. While the data in that column are of general interest in indicating the relative degree of loading at the various plants, they should not be interpreted too broadly for the reason that some of the B.O.D. data utilized in the calculations may have been obtained from grab samples while other data were obtained from composite samples.

Placing a Plant in Operation

Before going into the detailed operation control procedures, it might be well to discuss first the general method of placing a plant in operation. In building up a healthy, well conditioned activated sludge it is important that certain fundamental factors be considered and followed.

The organisms which go into the building of an activated sludge are all present in the sewage, but in order that those organisms may become acclimated and cultivated it is important that the aeration mixture contain dissolved oxygen and that the organisms be fed at a proper rate. In starting a plant, allow the aeration tanks to fill with settled sewage and start the aerators, setting them to operate continuously. Allow 30 to 50

*Published in *Sew. Wks. J.*, 17, 101 (1945). Reprinted by permission of the Federation of Sew. Wks. Assns.

TABLE I
General Information Regarding Contributing Plants.

Plant	Connected Population	Type Sewers	Average Flow (MGD.)	Average Primary Sedimentation Period (Hr.)	Aeration Units: Type Aerators	Aeration Units: Number	Aeration Units: Average Aeration Period (Hr.)	Aeration Units: Cu. ft. Aeration Tank Capacity per Lb. Primary Effl. B.O.D.	Average 5-day B.O.D (ppm.): Raw Sewage	Average 5-day B.O.D (ppm.): Primary Effluent	Average 5-day B.O.D (ppm.): Final Effluent	Final Tank Collector Speed (Ft. per Min.)
Batavia, Illinois	4,700	Combined	0.895	0.45	American	7	4.62	57	287	70	9	1.0
Belvidere, Illinois	9,000	Separate	1.202	0.99	Chicago	9	7.00	49	223	126	5	0.75
Birmingham, Michigan	12,500	Combined	2.60	0.68	American	8	4.05	78	81	52	9.7	—
Bryan, Ohio	5,000	Combined	0.682	2.00	Chicago	6	9.26	109	102	67	10	2.0
Buchanan, Michigan	4,500	Separate	0.787	—	Chicago	3	3.10	46	96	56	21	2.0
Bucyrus, Ohio	9,600	—	1.066	1.5	American	6	6.00	45	263	126	4	—
Carlstrom Field	Restricted	Separate	0.067	1.5-3.0	Chicago(1)	2	10-20	99	181	138	3.6	—
Coldwater, Michigan	1,200	Separate	0.154	3.1	American	3	10.40	126	165	82	4	1.5
Crystal Lake, Ill.	5,000	Separate	0.30	—	Chicago	4	9.90	87	240	125	6	11.8
Dorr Field	Restricted	Separate	0.053	3.75	Chicago(1)	2	25.06	131	190	131	13	—
East Lansing, Mich.	13,500	Combined	1.838	1.20	American	9	4.80	107	79	40	4	10.0
Elmhurst, Illinois	16,000	Combined	1.50	1.60	Chicago	12	6.90	57	123	103	13	—
Ferguson, Mo.	12,000	Separate	0.80	2.0	Chicago	5	5.70	95	110	50	6	1.5
Geneva, Illinois	5,500	Separate	0.90	1.4	Yeomans	9	6.00	29	325	170	11	1.0
Great Lakes, Ill.	Restricted	Separate	Restricted	2.0	Chicago	12	10.00	—	375	225	19	2.0
Greece, New York	10,060	Separate	0.599	1.95	American	8	8.80	39	231	170	17	4.2
Illinois Ordnance	Restricted	Separate	0.100	5.32	Yeomans	3	22.00	163	150	130	1.5-3.0	1.0
Kewanee, Illinois	8,000	Separate	0.860	2.2	Yeomans	12	7.40	55	289	119	11	1.5
Knightstown, Ind.	500	Separate	0.060	—	Chicago(1)	2	—	—	—	—	—	—
Ladd, Illinois	1,200	Combined	0.070	1.75	Chicago(1)	2	17.00	37	—	—	—	—
Lambert Field	Restricted	Separate	0.250	1.2	Chicago	3	7.50	35	200	176	20	—
Laurens, S. Car.	5,700	Separate	0.480	0.50	Chicago	4	9.20	—	900	—	40	2.0
Milton Junction, Wis.	2,000	Separate	0.080	12.0	Yeomans	2	25.00	87	800	300	20	1.0
Radnor—Haverford, Pa.	10,400	—	0.80	1.0	Yeomans	8	10.00	91	172	104	7	—
Riddle Field	Restricted	Separate	0.015	6.4	Chicago(1)	1	42.00	330	165	92	7	—
Rochelle, Illinois	4,400	Separate	0.416	2.5	American	6	9.90	72	249	148	12	4.0
Stevens Point, Wis.	14,000	Separate	1.41(2)	2.0	American	6	7.60	25	270	180	10	4.0
Waterloo, Illinois	1,300	Separate	0.098	—	Chicago	2	15.00	150	232	89	26	0.5
Willow Run	Restricted	Separate	1.49	3.0	Chicago	16	12.00	76	155	129	18	1.0

(1) Combination Aerator—Clarifier. (2) 60 per cent given complete treatment.

per cent of the design flow to pass into the aeration tanks, by-passing the remainder of the settled sewage.

The Chicago Pump Co. (Operation Bull. No. 1165) suggests by-passing all settled sewage for 24 hours after starting the aerators and before allowing additional sewage to pass through the aeration tanks. After the final settling tanks are full, start the collecting mechanism and return sludge pumps (or open valve on automatic sludge return line*). The return of these previously aerated sewage solids will cause a gradual increase in the mixed liquor solids concentration. Walker, of the American Well Works, in notes for a lecture on activated sludge plant operation presented before the 1944 Illinois Sewage Works Operators' Short Course, stated:

"The quantity of sewage added to the aerated mixture is gradually increased from day to day. The return to the incoming sewage of the settled solids (taken from the final settling tank) from the previously aerated sewage will gradually increase the solids concentration of the aerated mixture. The addition of more sewage has to be done very carefully so as not to supply more organic food to the organisms than they can consume. Overfeeding of the organisms will have a very detrimental effect upon their existence because there will then be more organic matter being absorbed by the young activated sludge than it can digest. In a like manner, underfeeding has a very detrimental effect and it can be easily imagined that starved organisms do not thrive.

"Operate the plant at ⅛ to ½ rated (design) load and do not try to force the early activated sludge development. As the activated sludge starts to develop (recognized first by much white foaming and characteristic strong, spicy odor), increase the sewage load slowly.

"Stop applying the sewage load at once if the activated sludge seems to be reverting back to the air flocculated solids phase." (Note: Walker divides activated sludge development into two phases: first, the air flocculated solids phase, and, second, the true activated sludge phase.)

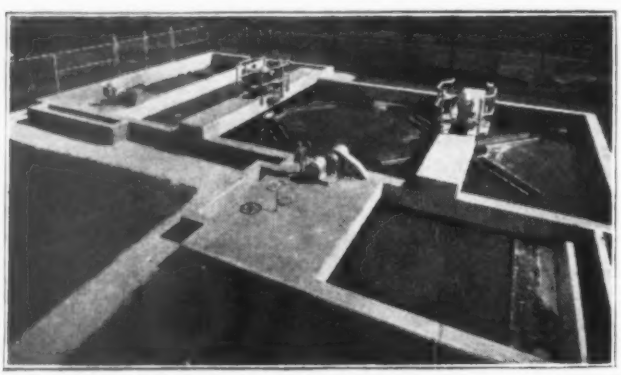
The mixed liquor solids concentration will gradually build up to a point where sufficient solids are present to permit turning the entire sewage flow into the aeration tanks. Operation Bull. No. 1165, of the Chicago Pump Co., suggests that this can be done when the suspended solids in the mixed liquor reach 400 ppm., and that careful wasting of activated sludge can be started when the suspended solids reach 600 to 800 ppm.

When starting a new activated sludge plant to serve a new sewer system it is nearly always found that the sewage flow is well below the design flow, which makes it advisable to begin operations, as described above, utilizing only a portion of the total aeration tank capacity of the plant. Similarly, if the plant is designed with more than one primary tank and final settling tank, it would appear desirable to start operations with only a portion of the settling capacity to avoid difficulties from overlong detention periods. Obviously, in the smaller plants this flexibility of operation of plant units is not common.

After the activated sludge has been developed, the operator finds that there are several variables over which he can and must exercise control in the operation of his plant. Those variables are:

1. Mixed liquor solids concentration.
2. Rate of returning sludge.
3. Application of air (amount of aeration).
4. Aeration period.
5. Condition of the activated sludge.

Those operation variables must be controlled by the operator to keep the plant in balance so that it can accommodate the normal loading received, and the same variables must also be regulated to accommodate fluctuations in loading, whether seasonal or otherwise. The comparison of control practices at the various plants contributing to this review clearly indicates the effect of local conditions upon control procedures.



American Well Works Mechanical Aeration Installation at McHenry, Ill.

*In the case of plants using American aerators.

Mixed Liquor Solids Concentration

The operation variable which usually receives first attention, and which is often considered the most important in activated sludge plant operation, is the mixed liquor solids concentration. The concentration of solids carried in the aeration tanks is adjusted by apportioning the activated sludge removed from the final settling tank between return and waste, this apportionment being accomplished in one of several ways, depending on the plant design.

In plants equipped for automatic sludge return (utilizing American aerators), wasting of activated sludge is usually controlled by a telescoping valve installed on the end of a sludge line branching off the return line and located in a waste sludge well. In plants employing pumps for returning sludge, the sludge is commonly pumped to a division box which can be manually adjusted to apportion the activated sludge between return and waste. In the case of the Chicago combination aerator-clarifier plant, a special waste activated sludge hopper is provided through which there is a circulation of mixed liquor at a controllable rate. Settling of waste activated sludge takes place in this hopper.

Data including the average and range of mixed liquor solids carried in the aeration tanks of 29 plants, together with the average sludge index and 30 minute mixed liquor settleable solids, are tabulated in Table II. These data indicate that fourteen of the plants carry average solids of 600 to 1,000 ppm., five plants carry average solids of 300 to 500 ppm., and one plant carries

TABLE II.

Mixed Liquor and Return Sludge Data

Plant	Mixed Liquor Suspended Solids (ppm.) Average	Mixed Liquor Suspended Solids (ppm.) Range	Return Sludge: Average Return Rate (%)	Return Sludge: Return Rate Range	Return Sludge: Average Suspended Solids (ppm.)	Average Sludge Index	Average Mixed Liquor Settleable Solids (% in 30 min.)
Batavia, Illinois	824	806- 851	30	25-45	1,519	56	4.6
Belvidere, Illinois	557	416- 903	17	16-19	—	643	36
Birmingham, Michigan	2,000	1,500-2,600	28	22-34	10,700	70	11
Bryan, Ohio	711	557- 780	30	25-45	2,037	603	57
Buchanan, Michigan	676	530- 882	25	—	—	266	18
Bucyrus, Ohio	1,000	—	25	25-50	4,800	150	20
Carlstrom Field	840	260-1,400	—	—	—	631	53
Coldwater, Michigan	1,876	1,550-2,080	50	42-78	4,583	122	23
Crystal Lake, Ill.	550	—	25	—	—	85	4-10
Dorr Field	744	467- 910	—	—	—	483	36
East Lansing, Mich.	1,100	500-1,790	33	23-44	2,700	53	12
Elmhurst, Illinois	433	315- 655	27.5	16-41	—	530	22
Ferguson, Mo.	300	200- 600	25	20-30	—	360	13
Geneva, Illinois	621	443- 778	14	13-17	2,518	110	5
Great Lakes, Ill.	750	700-1,000	50	40-55	2,500	280	10
Greece, New York	1,036	669-1,470	17	7-39	4,170	173	19
Illinois Ordnance	900	500-1,400	46	—	2,250	48	7
Kewanee, Illinois	1,062	200-2,800	30	—	3,453	309	30
Knightstown, Ind.	800	—	—	—	—	800	50
Ladd, Illinois	600	300-1,000	—	—	—	167	10
Lambert Field	500	—	25	—	900	700	35
Laurens, S. Car.	600	500- 700	20	—	700	350	36
Milton Junction, Wis.	200	—	50	—	—	1,000	20
Radnor—Haverford, Pa.	900	800-1,000	40	35-45	2,000	155	18
Riddle Field	786	—	—	—	—	114	9
Rochelle, Illinois	445	310- 534	61	—	1,248	405	18
Stevens Point, Wis.	600	500- 900	50	40-60	1,800	135	10
Waterloo, Illinois	641	—	33	—	758	410	27
Willow Run	1,100	650-1,550	28	16-56	3,400	237	40

an average of only 200 ppm. Six of the plants carry average solids of over 1,000 ppm., two of which (Birmingham, Mich., and the Michigan State Home at Coldwater) carry averages of 2,000 and 1,876 ppm., respectively.

Grinnell (Birmingham, Mich.) states that his plant is normally operating at nearly 100 per cent overload, which makes it necessary to carry as high a mixed liquor solids concentration and index as possible in order to obtain good treatment at the short aeration period. At concentrations above 2,600 ppm. difficulty is experienced with floc carrying over the weirs of the final settling tanks, and this problem is accentuated when the volatile solids in the mixed liquor exceed 75 per cent.

Grinnell further states that he attempts to maintain a sludge as high in per cent volatile solids and as slow in settling as the plant flow loading will allow without carrying floc over the weirs. The sludge high in volatile content is, of course, more active. This is important in an overloaded plant in which it is necessary to obtain the most from the sludge concentration which can be carried. Grinnell states that two intercepting sewers lead to his plant, one of which carries very dilute sewage following rains. It is usually necessary to by-pass this sewage, which is high in ash content (fixed solids) due to the silt washed off the gravel streets in the contributing area. If the volatile content of the mixed liquor is high when a rain occurs, the high ash content sewage is carried into the plant for a time to reduce these high volatile solids, thus to make the sludge more dense so that it will settle in the final tanks at the increased rate of flow. If loading of the sludge with the silt-bearing sewage is carried to a point where the mixed liquor solids are lowered to 50 per cent volatile, or less, Grinnell states that an insufficiently active sludge results, as might be expected.

Tompkins (Michigan State Home, Coldwater) indicates that best results are obtained at his plant with an average mixed liquor solids concentration of 1800-1900 ppm. Decker (Bryan, Ohio) indicates that best results are obtained there when the mixed liquor solids are carried at 800 ppm. He indicates that about 100 ppm. higher mixed liquor solids are usually carried on Monday, the local washday, to compensate for the additional load.

Lamb (Belvidere, Ill.) carries an average of 557 ppm. of mixed liquor solids. He prefers to operate with mixed liquor solids as low as possible since it has been his experience that, while the plant might be somewhat more sensitive to upset at the lower solids concentrations, it will recover from an upset much faster than at higher solids values. Patriarche (East Lansing, Mich.) indicates that best results are obtained there at concentrations of 1,100 ppm., but states that the concentrations vary over a rather wide range in spite of efforts to control them. That this difficulty is similarly experienced in many plants is indicated by the rather wide range of mixed liquor solids reported from nearly every plant.

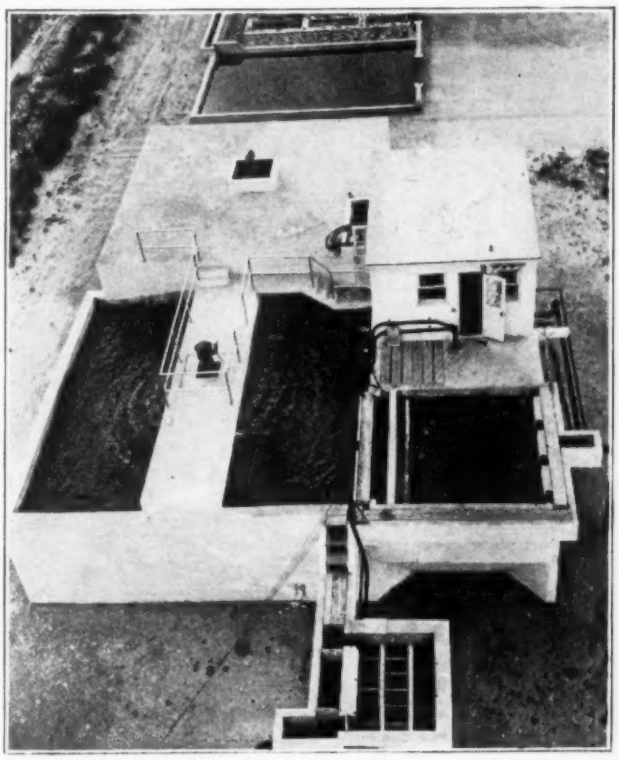
Denise (Greece, N. Y.) states that 800 ppm. of solids are carried during the summer months and about 1,100 ppm. during the winter months. It was not indicated whether a higher volatile solids content during the summer months enables lowering the mixed liquor solids, or whether there might be a drop in the load during the summer, which would appear rather unlikely. Carothers (Riddle Aeronautical Institute) indicates that his rule for mixed liquor solids is "the higher the better, commensurate with the oxygen supply."

Freeman (Geneva, Ill.) maintains approximately 620 ppm. of mixed liquor solids, but due to other duties which make it impossible to maintain as complete laboratory data as desired, mixed liquor solids are controlled on the basis of the settleable solids test, an effort being made to maintain between 5 and 8 per cent after 30 minutes. Lt. Comdr. Klay (Lambert Field, St. Louis) indicates that the settleable solids test on the mixed liquor is used to control the concentration, as does Bradley (Buchanan, Mich.) and Longley at Radnor-Haverford, Pa. Fischer (Kewanee, Ill.) attempts to maintain 800 to 1,000 ppm. of mixed liquor solids, and utilizes both suspended and settleable solids as a control basis for determining return sludge and wastage.

In general, the amount of mixed liquor solids which can be carried in a plant is limited only by the air supply and by certain hydraulic capacity factors. It is noteworthy here that mixed liquor solids concentrations as high as 2,600 ppm. are carried in one plant utilizing mechanical aerators, apparently without difficulty in supplying adequate aeration. The 2,600 ppm. of solids are carried at Birmingham, Mich., with operation of the aerators only 50 per cent of the time, and a dissolved oxygen content of 4.0 ppm. is maintained in the final aerator. It should be noted that the sewage at Birmingham is quite weak, however, an average aeration period of only 4.05 hr. being available. It appears that the solids concentration which can be carried in some mechanical activated sludge plants overlaps well into the range of solids usually associated with diffused air operation.

It is logical, from the standpoint of economic considerations, that no more mixed liquor solids be carried than necessary to meet the conditions under which a plant must operate and to permit obtaining the required effluent quality. Where the strength of sewage is widely variable, care must be exercised to maintain sufficient solids to handle the peak load. Most operators, particularly those operating plants treating widely variable sewages, must determine by intelligent trial and error methods the mixed liquor solids concentration which gives the best and most economical results, first emphasis being placed upon obtaining an effluent fitted to the outlet stream requirements.

The Chicago Pump Co. (Operation Bull. No. 1165) states that the operation of many plants utilizing Chicago mechanical aerators has indicated that for best results the mixed liquor solids should be from 600 to 1,000 ppm. Likewise, experience with their combination aerator-clarifier has indicated that best results are obtained in the 500 to 800 ppm. solids range. Yeomans Brothers suggests a range of mixed liquor solids from 300 to 1,000 ppm., with an average taken at about 500 ppm. They further point out the need for the experimentation to determine the optimum concentration for any given plant.



Chicago Pump Co. "Package" Plant at Consolidated Vultee Aircraft Corp., Miami, Fla.

At most of the mechanical activated sludge plants covered in this review, the suspended solids test is used as the basis for controlling the mixed liquor solids; however, at five of the plants it was indicated that the settleable solids determination is used for that purpose. Most of the plants reporting indicated that settleable solids in the mixed liquor are utilized largely as an indication of the general condition of the sludge and for computing the sludge index, and several operators referred to the difficulty in controlling a plant on the basis of settleable solids alone because of the variable settling characteristics of the sludge.

Cortner (Indiana Children's Home, Knightstown) utilizes a centrifuge in controlling mixed liquor solids, in lieu of suspended or settleable solids, as does Lovejoy at Laurens, S. C. In that regard, it should be stated that there are many plants utilizing centrifuge results on mixed liquor as a control test.

The bulletin on "The Operation of a Chicago Combination Aerator-Clarifier Plant" states:

"It has been found that a centrifuge, either hand operated or electric, is a big help in controlling the concentration of solids in the mixed liquor. If the character of the sludge changes, the settling tests may vary greatly without any great change in the weight of the solids present, or vice versa. The centrifuge readings vary only slightly, regardless of the character of the sludge, being nearly directly proportional to the weight of solids present.

The centrifuge cannot replace the balance for accuracy, but is close enough for control of the mixed liquor solids and is far more accurate than the settling test alone.

"A centrifuge comes with two 15 cc. graduated tubes. Fill these to the mark with a well mixed sample from the aerator and insert in the holders. Spin for 3 minutes at 1,800 rpm. (2 revolutions per second with the commonly used hand centrifuge). Let the centrifuge die down without braking and read the sludge volume in the bottom. A reading of 0.25 to 0.40 ml. gives the proper amount of solids. This is approximately the same as 500 to 800 ppm. suspended solids."

A study of the average mixed liquor settleable solids data for the various plants, as tabulated in Table II, indicates a wide variation in per cent settleable solids for sludges of approximately the same suspended solids concentration, which is further verification of the need for intelligent and careful use of settleable solids if it is to be depended on as the sole basis for controlling mixed liquor solids concentration.

As a general average, in plants treating normal domestic sewages not containing troublesome industrial wastes, it may be stated that with activated sludges of good settling characteristics, 30-minute settleable solids in the 10 to 20 per cent range are most common.

Summarizing briefly, the following comments regarding mixed liquor solids control are offered:

1. Concentrations of 600 to 1,100 ppm. appear to be most common, although some plants operate below that range, and local conditions require that some plants operate above it. It is noteworthy that two plants included in this review are carrying concentrations normally associated only with diffused air operation, and without difficulty in supplying adequate aeration.

2. At all but five of the plants here represented, mixed liquor solids are controlled by use of suspended solids or centrifuge results, those five making use of settleable solids. It is recognized that many plants operate entirely on the basis of settleable solids, which procedure requires alertness on the part of the operator in interpreting the changes which occur in sludge settleability.

3. As stated by Wisely in his review of diffused air operation (see *Water Works and Sewerage*, June, 1944), it is suggested that there be no hesitancy in varying the mixed liquor solids to meet seasonal variations in load and temperature.

4. There does not appear to be evidence of any trend among the mechanical activated sludge plant operators to utilize the volatile content of the mixed liquor solids as a control basis in lieu of the total suspended solids, only one operator having an overloaded plant serving a combined sewer system indicating the use of volatile solids as a control.

Rate of Returning Sludge

The return of settled activated sludge from the final settling tanks to the aeration tanks serves to maintain the necessary solids concentration in the mixed liquor, and keeps the activated sludge in a usable condition. Sludge which remains in the final tanks too long will become septic and, upon its return to the aeration tanks, will exert a high oxygen demand, will not perform its purification functions effectively, and may readily cause a complete plant upset. Too low a rate of return may result in a septic return sludge; conversely, too high a rate of return may presumably cause difficulty in disposing of the excess sludge, due to the larger volume which would have to be wasted of the resulting dilute sludge in order to maintain the mixed liquor solids at the desired concentration.

Walker (American Well Works), in the lecture notes previously quoted, listed the following causes of a septic return sludge:

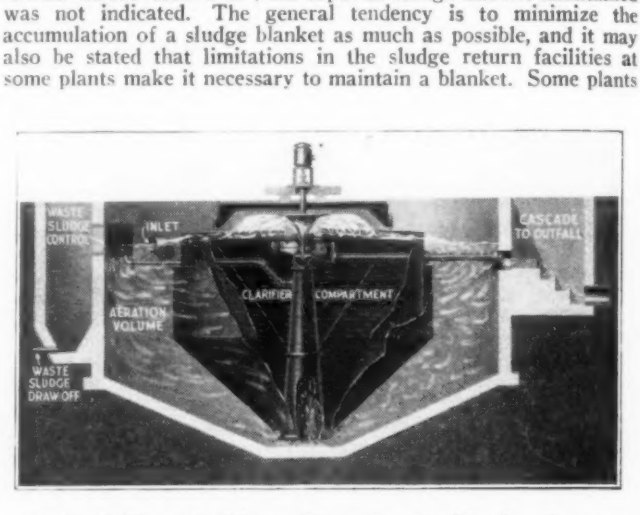
"1. The return pumping rate is not great enough and a blanket forms on the floor of the final tank, causing the activated sludge to be out of aeration too long.
"2. Faulty, slow or inefficient sludge conveying mechanisms in the final tanks.
"3. Bad draw-off hydraulics causing 'holes' to be pulled in the hoppers and leaving the sludge behind.
"4. Final tank units too large, making expeditious movement of settled sludge impossible.
"5. Not enough activated sludge available to treat the organic matter fed to it.
"6. Too much dense primary solid matter enmeshed in the activated sludge.
"Where return activated sludge is septic and reaeration is impossible, either increasing the aeration at the head end, or returning sludge at a number of points along the line of flow may prove helpful."

The rates of return sludge practiced in the plants contributing to this summary, together with the return sludge suspended solids data, are tabulated in Table II. Of the 24 plants indicating their return sludge rate, 12 employ rates between 20 and 30 per cent, eight employ rates between 30 and 50 per cent, one employs a rate of 61 per cent, and only three employ rates under 20 per cent. The data on the range of the return sludge rate indicate a considerable variation in the return sludge rate at a number of plants. Sludge return rates for the combination aerator-clarifier plants are not given, as the automatic return of sludge is a feature of that design.

The operation bulletin of the Chicago Pump Co. suggests that the rate of returning sludge should be at 20 to 25 per cent of the sewage flow, and warning is given against allowing the sludge to become septic at any point. Yeomans Brothers suggests a return sludge rate of 15 to 20 per cent, indicating it may be less than that, but they do not recommend a rate in excess of 20 per cent. Yeomans further suggests that the best rate of sludge return be determined by test so that the optimum mixed liquor solids concentration is maintained to produce the highest degree of treatment.

A study of the return sludge suspended solids data does not reveal any definite relationship between the concentration of return sludge and the rate of sludge return. Variations in sampling methods may explain this seeming inconsistency. It is generally expected that lower solids concentrations will result from the higher return sludge rates.

Of the plants commenting on the maintenance of a sludge blanket in the final settling tanks, 14 plants do not maintain a blanket, but seven plants indicate that a sludge blanket is maintained. In the latter cases, the depth of sludge blanket maintained was not indicated. The general tendency is to minimize the accumulation of a sludge blanket as much as possible, and it may also be stated that limitations in the sludge return facilities at some plants make it necessary to maintain a blanket. Some plants do maintain some sludge blanket so that this sludge will be available for shock loads; however, that practice was not referred to specifically by any of the plants contributing to this summary.



Sectional View of Chicago Pump Co. Combination Aerator-Clarifier.

Application of Air

In diffused air activated sludge plant operation, the air must serve both as a mixing agent and to maintain the aerobic conditions required for effective operation. In a mechanical activated sludge plant, the air serves to satisfy the oxygen requirements of the activated sludge-sewage mixture and to maintain aerobic conditions, the mixing being accomplished by the mechanical aerator as it circulates the contents of the aeration tank. It is by means of this mechanical circulation that the conditions requisite for the entrainment of air in the mixed liquor are obtained. In a mechanical activated sludge plant the total quantity of air available is governed by the number of aerators in use and by the basic oxygenation capacity of the aerators, which is somewhat variable by means of adjustment of the equipment.

Certain of the Yeomans Brothers-Simplex aerators are equipped so that the amount of air applied can be varied by a simple adjustment of the speed of rotation of the aerator cone. Speed regulation from 30 per cent below to 30 per cent above normal speed is commonly provided.

The aerators of the Chicago Pump Co. are rated at about 6 ppm. of oxygen supplied to the entire tank contents per tank turnover, the number of tank turnovers per hour determining the basic oxygenation capacity rating. In an existing installation, the oxygenation capacity of Chicago aerators can be increased about 25 per cent by increasing the aeration propeller size, and about 150 per cent by increasing both propeller and motor sizes, according to their Bulletin 128-L.

The American Well Works aerators are of the injector plate and mixing well type, or of the type equipped solely with air intake tubes. In the well type, the water level in the well can be regulated by adjusting the openings in the injector plate, which also regulates the air input. In the air tube type, the quantity

of air applied can be readily adjusted by opening or closing the orifice unit on the air intake tubes.

For flexibility and economy of operation, the majority of mechanical activated sludge plants are equipped with time-clock control for the aerators which, by means of a simple adjustment, allows any desired operation-rest cycle during each cycle of the clock. The clock cycle is usually 15 or 20 minutes. The sewage strength at the majority of plants varies considerably, and for that reason it would not be economical to operate the aerators continuously at the same rate required for strong sewage at those times when a dilute sewage is received for treatment. In adjusting the time-clock control for the aerators the operator is nearly always guided by the dissolved oxygen content of the mixed liquor, most operators attempting to maintain a definite oxygen threshold before resorting to time-clock control.

TABLE III
Aerator Operation and Dissolved Oxygen Control

Plant	Average Operation of Aerators, Per Cent of Time	Mixed Liquor D.O. (ppm.) Inlet	Mid-point	Outlet	Final Effluent D.O. (ppm.)
Batavia, Illinois	76	1.0	2.5	5.0	3.2
Belvidere, Illinois	(1)	—	—	3.4	5.1
Birmingham, Michigan	50	—	—	4.0	2.0
Bryan, Ohio	50-100	0.1	2.0	3+	1.3
Buchanan, Michigan	100	—	—	5.5	—
Bucyrus, Ohio	85	—	—	—	5.6
Carlstrom Field	90	2.1	2.1	2.1	3.7
Coldwater, Michigan	90	—	—	6.0	5.0
Crystal Lake, Ill.	85	—	—	—	3.0
Dorr Field	90	2.7	—	—	2.9
East Lansing, Mich.	50-100	2.2	3.5	5.2	4.6
Elmhurst, Illinois	100	—	—	3.0	6.0
Ferguson, Mo.	65	—	—	4.5	5.0
Geneva, Illinois	100	0.2	2.2	3.0	4.6
Great Lakes, Ill.	80	2.0	—	1.5	0.2
Greece, New York	100	—	—	4.4	2.6
Illinois Ordnance	40	2.0	3-4	4-5	6.0
Kewanee, Illinois	77	—	—	—	2.7
Knightstown, Ind.	100	—	—	—	6.0
Ladd, Illinois	50	4.0	4-5	6.2	6.0
Lambert Field	62	—	—	3.0	2.0
Laurens, S. Car.	100	3.5	—	4.0	2.5
Milton Junction, Wis.	75	—	—	2.0	0.4-1.0
Radnor—Haverford, Pa.	100	1.7	2.8	5.0	4.6
Riddle Field	70	—	1.8	—	4.4
Rochelle, Illinois	100	1.8	4.0	6.6	4.1
Stevens Point, Wis.	66	—	—	0.3	0.8
Waterloo, Illinois	95	3-4	4.1	4.9	4.7
Willow Run	100	2.7	2.3	3.7	1.7

(1) Operated according to oxygen demand curve.

Table III indicates the average per cent of time that the aerators are in operation at the various plants, and also indicates average dissolved oxygen values in the mixed liquor at the inlet, mid-point and outlet of the aeration tank battery.

Decker (Bryan, Ohio) attempts to maintain 0.5 to 1.0 ppm. as a minimum dissolved oxygen concentration in regulating his time-clock control of aeration. Olson (Batavia, Ill.) has the last four of his six operating aerators on time-clock control, and he attempts to maintain a D.O. of 4.0 ppm. in the final aerator to meet shock loads of milk wastes. Grinnell (Birmingham, Mich.) operates his aerators to maintain a minimum D.O. of about 2.0 ppm. in the final effluent, which gives an average of 4.0 ppm. in the final aerator. The sewage at Birmingham is quite weak, and although 2,600 ppm. of mixed liquor suspended solids are carried at times, the operation of the aerators 50 per cent of the time has proved adequate.

Lamb, at Belvidere, Ill., made some rather extensive oxygen demand studies by means of the Odeeometer and determined a typical oxygen demand curve for the conditions encountered at Belvidere. He states:

"The curve shows that at Belvidere large amounts of air are needed to satisfy the demand during the first 15 minutes of aeration period, and that at least a 7-hour aeration period is needed. By operating aeration units in accordance with this curve, we have obtained excellent operation with economy. Before operating in accordance with this curve, tests showed that there was considerable variation in the oxygen demand of the return sludge. On many occasions this demand was considerably higher than could be furnished by the aeration units.

"Tests taken after operating in accordance with the oxygen demand curve show that operation by this method eliminates excessive oxygen demand of the return sludge, and keeps the demand down where the aerators can meet it.

"The economy of operation is attested by the fact that power consumption has been reduced from a previous figure of 0.371 kw. hr. per lb. B.O.D. removed in 1938 to 0.293 in 1939, 0.281 in 1940, and 0.257 in 1941.

"Aerators are cut in and out of service and time-clock operation used in order to meet the changing oxygen demand brought about by changes in either the volume or strength of the sewage." (Note: A complete description of the experimental work done by Lamb, and its interpretation in plant operation, is given in his thesis, "The Operation of an Activated Sludge Treatment Works with Special Reference to the Treatment of Cannery Wastes and the Oxygen Demand of Sewage and Activated Sludge Mixtures," University of Illinois Graduate School, 1940.)

Lamb further states that dissolved oxygen tests are made twice daily on the mixed liquor in the final aerators in each of the three batteries, and a low D.O. in the evening followed by a substantial recovery the next morning indicates economical operation in accordance with the oxygen demand curve.

Howe (Illinois Ordnance Plant) operates the aerators a sufficient time to produce an effluent in which the dissolved oxygen content is as great or greater than the B.O.D. of the same effluent. The Illinois Ordnance Plant is operating considerably below the design flow, and as a result the lift stations operate at irregular intervals causing the sewage flow to be received more or less as shock loads. Due to the long aeration period available, however, operation of the aerators during 40 per cent of a time-clock cycle has proved adequate to maintain a high degree of treatment.

Patriarche (East Lansing) finds that as long as a slight amount of dissolved oxygen can be maintained in the return sludge, his plant is operating satisfactorily. A dilute sewage during the spring months makes 50 per cent time-clock operation of the aerators adequate, but nearly 100 per cent operation is considered necessary during the summer. An average D.O. of 5.2 ppm. is maintained in the final aerator. Patriarche states that a tapered aeration schedule has been tried, but that better results are obtained by operating all aerators the same percentage of time.

Yeomans-Simplex Mechanical "Aerifier" at Gulf Shores, Fla.

Denise (Greece, N. Y.) indicates that about 2.2 hours' preaeration of the primary effluent is standard operation, the sewage leaving the preaerator having a D.O. of 1.5 to 3.0 ppm. One hundred per cent operation of the aerators maintains a minimum of 4.0 ppm. D.O. in the final aerator. Tanari (Ladd, Ill.) indicates that from 4.0 to 6.0 ppm. D.O. seems to be necessary to prevent bulking, which is probably caused by milk wastes.

Wolf (Waterloo, Ill.) attempts to maintain at least 3.0 ppm. of dissolved oxygen before resorting to time-clock control, as does Tompkins at the Michigan State Home. Tompkins has one time-clock for controlling the last two of the three aerators, the first aerator being run 100 per cent of the time. Lt. Comdr. Klay (Lambert Field) indicates that an average D.O. of 3.0 ppm. is maintained in the final aerator.

Fischer (Kewanee, Ill.) has operated on a tapered aeration schedule, operating four aerators 100 per cent, four at 75 per cent and four at 50 per cent of the time. Lovejoy (Laurens, S. C.) indicates that his plant was designed for tapered aeration, consequently all aerators are run continuously.

Miller, at Great Lakes, operates the mechanical aerators 80 per cent of each clock cycle. Due to the exceptionally strong sewage at Great Lakes it was found necessary to supplement the mechanical aeration with diffused air at approximately 1 cu. ft. per gallon, equivalent to about 490 cu. ft. per pound of B.O.D. removed. The supplemental diffused air system was designed for tapered aeration. Dulmage (Willow Run Bomber Plant) indicates that 100 per cent operation of the aerators has been necessary to maintain sufficient dissolved oxygen in the aeration tanks.

Several operators referred to the need for building up dissolved oxygen in the mixed liquor to meet shock loads.

Aeration Period

Stronger sewages require longer aeration periods in order to obtain a high degree of treatment, although some adjustment can be made by varying the mixed liquor solids within the limits of the air supply. Adjustments in the aeration period are made by placing aeration tanks in or out of service.

Table I lists the average aeration periods for the contributing plants. A wide variation in available plant capacity is indicated. In studying the aeration periods and sewage strengths indicated, it is not surprising that a considerable variation in operation control practices exists.

At six of the plants reporting, aeration periods between 3 and 6 hours are indicated; at six plants aeration periods in the 6 to 9 hour range are reported; at six plants aeration periods are in the 9 to 12 hour range. The remaining plants have aeration periods in excess of 12 hours. One plant, which is a combination aerator-clarifier plant operating far below capacity, actually has a 42-hour aeration period. Carothers, at that plant (Riddle Field, Clewiston, Fla.), indicates some difficulty with rising sludge in the final tank due to overaeration.

Few of the plants included in this review indicated that the aeration period is used as an operation control. Lamb (Belvidere) does place aeration tanks in and out of service as required by the sewage flow and strength, as does Freeman (Geneva). Denise (Greece) used four or five of the available eight aeration tanks during 1943 to maintain an average aeration period of 8.8 hours.

None of the plants indicated that reaeration of return sludge is practiced, although Decker (Bryan) suggested that some aeration of return sludge is effected in his plant by the unusual design, which provides a seven-foot fall from the return sludge draw-off pipe, and two additional cataracts of five feet from the sludge division box.

Normally, plants having adequate aeration periods have less difficulty maintaining a well conditioned activated sludge and, therefore, are less affected by shock loads than plants which must operate with a low aeration period, low to the extent that it must be compensated for by carrying higher mixed liquor solids.

Condition of the Sludge

The operator is constantly guided by the condition of the activated sludge in controlling the operation variables, and the maintenance of a well conditioned sludge reflects the success of the operation procedures, although at times the activated sludge assumes characteristics which can be controlled with extreme difficulty if at all. In determining the condition of the activated sludge, the operator is most commonly guided by some or all of the following observations:

1. Physical appearance, both in the aeration tank and in the laboratory, and odor.
2. Settling characteristics, and sludge index.
3. Microscopic examination.
4. Sludge activity, often referred to as oxygen demand, as determined by the Odeeometer or by direct chemical methods.

Appearance and Odor.—A well conditioned activated sludge is usually golden to moderate brown in color, has a pronounced, spicy (sometimes described as earthy) odor, and, upon becoming septic, assumes a sour odor and a gray to black color. A sample of a well conditioned activated sludge, when allowed to settle in a graduate, will usually become gas lifted to the surface in a few hours.

A well conditioned activated sludge, when placed in a glass cylinder, will quickly form itself into clearly defined flocs and then settle readily. As the sludge mass begins to settle, an upward movement of some flocs within the settling sludge mass is usually apparent as other flocs subside.

An experienced operator can usually determine from the appearance of the mixed liquor in the aeration tanks, to some extent at least, whether a well conditioned sludge is being maintained. In that regard, the Operation Bulletin of the American Well Works suggests:

"Another way of judging the condition of the aeration tank is to notice if the tank has a lively, active look while the air diffusion unit is running. Foam should be plentiful and white, and an active bubble pattern should form. When the machine is stopped, the sludge should rapidly crease and form furls. These furls should keep constantly changing and moving. If the sludge is in poor condition, the tank will have a dead look while the machine is running. There will be little or no foam present and the color of the sludge will change from brown to a pasty gray. When the machine is stopped the sludge will lie with an even surface and there will be no immediate creasing and furling."

Settling Characteristics and Sludge Index.—The success of activated sludge plant operation depends largely on maintaining the settleability of the activated sludge since the ready separation of the liquid and the sludge in the final sedimentation units is the final prerequisite to the production of a good effluent. Disregarding activated sludges affected by industrial wastes, thirty-minute settleable solids in the 10 to 20 per cent range are most common. Values above and below that range are also quite common, as the data in Table II would indicate.

The sludge index (Mohlman), commonly defined as the volume in mililiters occupied by one gram of sludge after thirty minutes' settling, is an expression of the density of the activated sludge and is found to be a helpful guide in plant operation. A rising sludge index may indicate underaeration, an overload, or a bulking tendency. The wide variation in average sludge index among the plants contributing to this summary, as listed in Table II, is most interesting and indicates the importance of determining the optimum operating range for any given plant. An index attendant to normal operation at one plant may occur during impending or actual bulking at another plant.

Microscopic Examination.—The use of the microscope in observing activated sludge condition is reported by five plants. Longley (Radnor-Haverford) reports weekly observations to check for *Sphaerotilus natans*. Miller (Great Lakes) makes daily microscopic examinations of mixed liquor and return sludge to check for *Sphaerotilus* and to observe the activity and types of organisms. Dulmage (Willow Run Bomber Plant) uses the microscope to check the general appearance of the sludge as a parameter of sludge condition. Occasional observations are made at Kewanee, Ill., and at least weekly at Michigan State Home.

Sludge Activity.—The oxygen utilization rate of the sludge in ppm. per hour, commonly referred to as sludge activity or oxygen demand, as determined by the Odeeometer or by direct chemical methods, is not utilized in most mechanical activated sludge plants. Of the plants included in this review, only Lamb at Belvidere has made use of extensive oxygen demand studies in plant operation, as previously recounted in this review.

Operation Problems

Bulking of Activated Sludge.—Nearly every activated sludge plant has at some time experienced a bulking condition of the activated sludge which is characterized by the loss of sludge settleability to such an extent that natural tank currents carry the sludge over the weirs. Bulking tendencies naturally occur with varying severity, and bulking may follow any condition from a mild expansion of the activated sludge to one in which there is little or no settlement of the sludge. Whether or not bulking will occur with any given degree of loss of sludge settleability will depend upon final settling tank flow velocities. At some plants, overloaded as to flow rate, a loss of solids occurs which is the direct result of excessive overflow rates in final sedimentation units. This condition simulates bulking but is not related to activated sludge condition.

Of the plants represented here, eight reported little or no trouble, two gave no reply, and the remainder indicated bulking difficulties with summarized causes as follows:

1. Shock loads from industrial wastes reportedly account for bulking at eight plants. Milk wastes and wastes containing excessive grease content were the major reported causes of bulking attributed to industrial wastes. The plant at Laurens, S. C., reports bulking due to shock loads of starch and dye wastes from a textile industry.

2. Septic sewage was listed as a cause of bulking at five plants, three of which indicated this difficulty usually follows a rain which flushes accumulated septic sewage solids from combined sewers. Difficulty with the carryover of waste activated sludge back to the aeration tanks was listed as a cause of bulking at several plants. Bulking sometimes follows the discharge of heavy supernatant liquor to the raw sewage at a number of plants.

3. The loss of operation balance was referred to by Bradley (Buchanan) and Patriarche (East Lansing) as basic causes of bulking. Patriarche specifically referred to bulking as resulting from too high or too low mixed liquor solids. Inadequate aeration might also be a cause.

4. It is common for overloaded plants to experience chronic bulking which defies the usual remedies. In cases of consistent organic overloading, unless the cause lends itself to ready reduction through pre-treatment or other practices, plant expansion is usually indicated for chronic bulking elimination.

5. High pH in the mixed liquor was attributed by Carothers (Riddle Aeronautical Institute) as a cause of bulking at a plant serving a military camp having a lime-softened water supply.

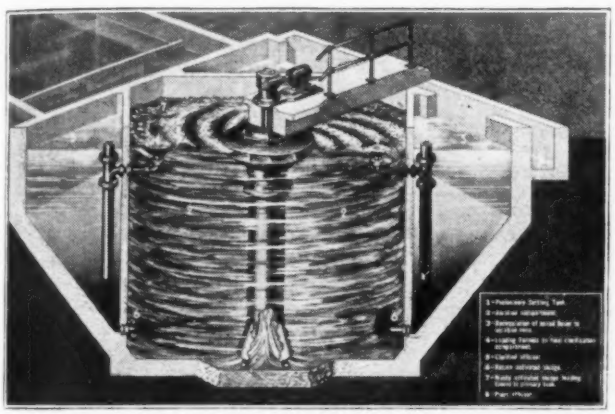
Continuous, careful attention to the operation variables will prevent many cases of bulking, as will the exercise of careful and intelligent control over industrial wastes discharging to the sewers. Requiring industry to provide pretreatment of troublesome wastes, removal of toxic wastes, and/or flow equalization

equipment to assure more uniform rates of discharge to the sewers often will minimize industrial waste influences.

If shock loads are a known cause of mulking, and if the loads are of such intensity that compensation or adjustment of operation variables may prevent or minimize some such cases of bulking, then it is obviously advantageous to learn to recognize promptly the receipt of a shock load so that preventive adjustments in operation may be made immediately. Air application and return sludge rate are commonly increased to meet shock loads. Either or both visual observations and analytical determinations may serve to indicate receipt of shock loads, depending on the nature of the wastes involved.

A common practice among operators upon the first evidence of bulking is to increase the application of air and the return sludge rate. If possible, reducing the load applied to the aeration tanks is a helpful adjunct since the intensity of loading is normally closely associated with bulking tendencies. Dick (Elmhurst, Ill.) indicates bulking control by increasing rate of return sludge and sludge wastage (probably applied after bulking is in progress).

Chlorination of the return sludge is utilized at Great Lakes, Batavia, Bucyrus, and Radnor-Haverford as a remedy for bulking. Miller (Great Lakes) begins chlorination of the return sludge at the first evidence of impending bulking, a dosage of about 4 ppm., based on raw sewage flow being utilized. Longley (Radnor-Haverford) indicates that bulking occurs frequently, but reports excellent control by chlorination of the return sludge at a dosage of 10 ppm., based on the return sludge flow.



Sectional View of Yeomans-Simplex "Aerifier."

Olson (Batavia) reports varied success in bulking control with the use of copper sulfate, and Todd (Milton Junction, Wis.) reports some success with the use of lime. Patriarche (East Lansing) controls bulking by the careful discharge of heavy supernatant liquor (20,000 to 40,000 ppm. suspended solids) direct to the aerators, at the same time increasing the application of air, if possible. This method is used at the first signs of bulking as a preventive measure rather than a cure, and Patriarche reports that, if done carefully, it has always been very effective. A well conditioned activated sludge is usually restored, at East Lansing, by this method in about two days. Patriarche cautions that the mixed liquor solids concentration must be carefully watched.

Denise (Greece, N. Y.) reports that no bulking has occurred there since the practice of aerating the primary effluent prior to adding the return activated sludge was established. An average aeration period of 2.2 hours is utilized for that purpose, and the primary effluent contains 1.5 to 3 ppm. of dissolved oxygen when the return activated sludge is added. This procedure freshens the sewage and reduces its oxygen demand, and is most helpful in cases where a septic sewage is received. Preaeration before primary sedimentation of a septic sewage would probably be of greater overall benefit; however, few plants are so equipped.

Carothers (Riddle Aeronautical Institute) indicates that bulking was attributed to too high a pH in the sewage at a military camp in Florida. He indicates that this pH is lowered in the primary tanks from 9.5 to 7.8-8.1 by merely stopping sludge pumping from those tanks, the drop in pH being attributed to the bubbling of the CO_2 gas, formed in the settled sludge, through the sewage flowing through the primary tanks. He indicates a content of 15 to 20 ppm. free carbon dioxide in the primary effluent. The sludge is skimmed off and pumped to the digester as soon as it is gas-lifted to the surface.

The problem at this plant is an unusual one, and the operation procedures indicated by Carothers are unusual and certainly unorthodox. An unusual combination of circumstances must exist to permit the successful use of the practices described by him, and high prevailing temperatures may be a factor. Primary tank efficiencies at Dorr and Carlstrom Fields are low, as would be expected as the result of the practices described by Carothers; however, it should be observed that available aeration periods are very long.

To restore the condition of a bulked activated sludge, common practice is to waste all or part of the bulky sludge, the amount wasted usually depending on whether or not sludge wasting and disposal facilities are limited. Denise (Greece) reports that prior to the utilization of preaeration of the primary effluent, when bulking occurred it was controlled by wasting down to 400 ppm. suspended solids in the mixed liquor and building up again to the normal concentration.

Rising Sludge in Final Clarifiers.—Rising sludge, a condition in which large masses or clouds of sludge rise to the surface of the final settling tanks, is attributed to a number of causes by the operators contributing to this summary.

Rising sludge attributed to overaeration occurs at Waterloo, Dorr Field, Riddle Field, Laurens and Rochelle. At the latter two plants the rising sludge condition follows a drop in the industrial wastes load, and Lovejoy, at Laurens, wastes sludge as a control measure. This condition is rather common at plants handling fluctuating industrial wastes loads, and it may occur at any plant when the load falls off considerably. Decreasing air application and reducing mixed liquor solids may prove helpful; however, when reducing mixed liquor solids it should be borne in mind that the heavier loadings can be anticipated to return.

It is difficult, and often impracticable, to adjust mixed liquor solids to load variations closely enough to prevent some degree of rising sludge from developing. The rising sludge condition which sometimes occurs when nitrification is high is commonly attributed to the reduction of nitrates in the sludge, the liberated nitrogen bubbles buoying the sludge masses to the surface. High nitrification commonly accompanies overaeration of the sludge, as does the presence of pin point floc in the plant effluent.

A condition, termed as rising sludge by the operators, occurs during the peak flows at Bryan, Batavia, Birmingham, Belvidere and Willow Run. This condition is probably closely associated with overflow rates and final settling tank velocities. Decker (Bryan) controls this condition by checking the sewage flow and backing up sewage in the outfall sewer during the peak flow. Lamb (Belvidere) indicates some success by increasing the return sludge rate. Dulmage, at Willow Run, has had some success in controlling this condition by discharging filtrate from the sludge filter into the inlet channel of the final settling tanks.

Septic conditions in the final settling tank sludge blanket will often result in the rising of masses of sludge to the surface. This was specifically referred to by Todd (Milton Junction), and is a basic reason why the majority of plants do not maintain a sludge blanket. This condition might occur if the final tank sludge collection mechanism was improperly installed or has been damaged.

Grease and Oil.—The receipt of undue quantities of grease or oil can seriously affect activated sludge plant operation. Six of the plants contributing to this review indicated some operation difficulty from oil or grease.

Grease may affect operation by acting as an organic overload, or by directly damaging the activated sludge into which it is readily absorbed. Impregnation with grease seems to reduce the oxygen absorbing powers of activated sludge, thus limiting the oxidizing capacity of the sludge which is reflected in the final effluent quality. This effect may be more pronounced when oil is received in quantity.

In lecture notes previously referred to, Walker (American Well Works) stated with reference to grease:

"Grease passing into the aeration tanks will be readily adsorbed by activated sludge. This adsorption is accumulative since the activated sludge cannot rapidly convert the grease adsorbed. Very soon (depending on the amount of grease and the amount of activated sludge) the activated sludge becomes composed of one-half or more of grease, and goes septic and useless, even though completely aerobic conditions surround the floc. This is a case of putrefaction going on internally in the sludge floc even while a residual of dissolved oxygen exists in the surrounding liquor.

"When the activated sludge has adsorbed too much grease, it loses its spicy odor and other activated sludge characteristics. It will turn to a light gray or black color and will refuse to settle well in the final settling tank. The effluent will develop a suspended turbidity which will be obvious in the clarity test. This increase in turbidity (diminishing final clarity) generally precedes the poor settling condition and is nearly always a good danger flag to the operator that all is not well."

Primary sludges or activated sludges high in grease content usually do not concentrate well, resulting in a digester operation problem at some plants. In addition to other effects, the pumping of a thin sludge to the digester results in a correspondingly larger quantity of supernatant liquor, which often creates a difficult disposal problem.

Waste Activated Sludge Disposal.—Of the plants contributing to this summary, eleven reported difficulty with waste activated sludge disposal, nine reported no difficulty, and the remainder indicated the method of disposal without further comment.

The most common method of handling waste activated sludge is to discharge it to the raw sewage so that the waste solids can be removed to the digester, along with the primary solids, after concentration in the primary settling tanks. That the method is not always satisfactory, however, is attested by the fact that of twenty plants utilizing it, eleven indicated that difficulties are encountered, and several others indicated that it must be practiced with care.

Failure of the waste activated sludge to settle out in the primary tanks during peak flows, thus imposing a recirculating load on the aeration tanks, was indicated at East Lansing, Lambert Field and Radnor-Haverford. Olson (Batavia) indicates that difficulty is most pronounced at times when a septic raw sewage is received. Browne (Bucyrus) indicates the waste sludge does not settle readily in the primary tanks when it contains appreciable grease and during the dry weather months (probably associated with the high volatile solids commonly prevalent during such periods).

Dulmage (Willow Run) wastes activated sludge continuously to the primary tanks, and adds milk of lime to the waste sludge at times to prevent its carryover into the aeration tanks. Enough lime is added to raise the pH of the waste sludge to a maximum of 9.0, and the method has met with some success. Wolf (Waterloo) and Krueger (Ferguson) refer to the necessity for frequent pumping of the raw-waste sludge mixture from the primary tanks. Failure to do so often results in septicity of the sludge mixture, accompanied by rising sludge masses in the primary tanks.

Several operators referred to the importance of wasting activated sludge at a low, uniform rate, particularly if primary sedimentation periods are low. Lamb (Belvidere) wastes sludge during the night when the sewage flow rate is low. Decker (Bryan) wastes sludge to the primary tanks continuously at a low rate and he indicates that, although a rather dilute mixed sludge results, less difficulty is encountered than when a concentration box was used for the waste sludge. Denise (Greece) indicates that the raw-waste sludge mixture is usually not well concentrated when pumped to storage or thickener tanks prior to vacuum filtration and incineration.

Howe (Illinois Ordnance) indicates that waste activated sludge is pumped to a concentration well and thence to the digester. This practice is unusually successful at that plant, as indicated by the 3 to 4 per cent sludges reported to obtain. Activated sludge wasting through primary settling tanks is often a problem when the sludge indicates a bulking tendency, or is actively bulking, as the waste sludge may not settle, thus imposing a recirculating load on the aeration tanks. This condition may greatly hamper efforts to restore activated sludge condition, since common practice is to increase greatly activated sludge wasting when bulking is out of control. Difficulties arise with nearly all methods of waste sludge disposal when bulking occurs.

Chironomus Larvae.—Although most of the plants contributing to this summary indicated that the *chironomus* fly larvae, more commonly called the bloodworm, have not been a problem, three of the plants reported that the larvae have caused interference with operation. At Michigan State Home, Tompkins reports the bloodworms constitute his most troublesome operation problem, and on occasion they have completely halted plant operation.

The larvae of the *chironomus* fly are bright red worms, commonly reaching lengths of one-half to one inch. The average life cycle of the fly is reportedly about five weeks, and under favorable conditions they multiply rapidly. The larvae surround themselves with sheltering cocoons of whatever material is available, and when activated sludge plants are infested the cocoon is formed of the activated sludge. The cocoons of sludge commonly adhere to tank walls for a while, then break loose and float on the final settling tank surface. The cocoons may break up and result in the formation of a thin layer of sludge on the tank surface. The heavy infestation of a plant may result in a serious loss of activated sludge, and in some plants the sludge has virtually disappeared overnight.

Tompkins (Mich. State Home) indicates screens have been placed over channels and quiescent portions of the final settling tanks to keep the flies out. He also indicates some benefit by maintaining a film of oil wherever possible. Denise (Greece) reported that difficulty with *chironomus* larvae was experienced during initial operation, and the only successful control was to clean out the entire plant and start over.

Pyrethrum, copper sulfate, and chlorine have been tried as control measures, usually with indifferent results. It is commonly conceded that dewatering and scrubbing infested tanks is the most effective control.

The *chironomus* larvae may be present in limited numbers without detrimentally affecting operation; however, they may readily develop into a major problem and their presence is always cause for concern.

Supernatant Liquor Disposal.—Probably the most common method of disposing of supernatant liquor is to return it to the raw sewage. This method of disposal is utilized at thirteen plants contributing to this review, with several of those plants indicating that alternate methods of disposal are available.

Olson (Batavia) reports this method as "a nasty practice and one difficult to control." Howe (Ill. Ordnance Plant) reports no difficulty when supernatant liquor is drawn slowly and carefully to the raw sewage. The supernatant liquor at Illinois Ordnance averages 800 to 1000 ppm., B.O.D., and only 110 ppm. suspended solids—a good supernatant at any plant. Tompkins (Mich. State Home) discharges a supernatant having an average of 619 ppm. suspended solids to the raw sewage without difficulty.

A supernatant containing 661 ppm. suspended solids is similarly disposed of without difficulty by Dulmage at Willow Run. Freeman (Geneva) reports that a supernatant having an average B.O.D. of 610 ppm. and suspended solids of 1730 ppm. is disposed of satisfactorily to the raw sewage if drawn slowly. Bulking results at Geneva if the supernatant liquor is drawn to the raw sewage at too high a rate.

Four plants reported disposal of supernatant liquor direct to the aeration tanks. Patriarche (East Lansing) refers to the necessity for carefully watching the mixed liquor solids concentration when utilizing that method of supernatant disposal. Considerable use is made of supernatant liquor at East Lansing to regulate and control the mixed liquor. This is interesting, particularly when considering that the supernatant liquor suspended solids average 25,000 ppm. Alternate disposal to the sludge drying beds is available.

Fischer (Kewanee) draws supernatant liquor into an idle aeration tank for storage and then discharges it to an operating aeration unit at a controlled rate. He states that this method has not proved too satisfactory. Dulmage (Willow Run) draws supernatant either to the aeration tanks or to the raw sewage. He cautions that supernatant liquor disposal to the aeration tanks must be done with care.

Browne (Bucyrus) indicates that when disposing of supernatant liquor to the aeration tanks, it is discharged at a low, continuous rate near the middle of an aeration tank battery. When supernatant liquor quality indicates it cannot be discharged direct to the aeration tanks at Bucyrus, it is discharged to a supernatant treatment tank equipped with an American Well aerator, this tank being operated on a fill-and-draw basis. The supernatant liquor is aerated, and coagulants and chlorine may be added if necessary. After this treatment, the settled sludge is returned to the digester and the clarified supernatant is withdrawn to the raw sewage. Browne reports very successful use of that treatment method.

A Pacific Flush Tank "atomizing" aeration-settling unit has been installed at Great Lakes for supernatant liquor treatment. Operating results were not yet available, as the installation was just completed and was undergoing minor adjustments when the questionnaires for this summary were distributed. Similar units at other plants have performed well, according to published data. Decker (Bryan) found it possible, through valve manipulations, to draw supernatant liquor into an idle flash mixing tank, where it is aerated and then discharged to the raw sewage during the night and low flow period. The supernatant liquor is drawn to the flash mixing tank at a slow rate.

At Belvidere, Indiana Soldiers and Sailors Children's Home, Dorr Field, Carlstrom Field, and Ferguson, supernatant liquor is disposed of by discharge of sludge drying beds. Lamb (Belvidere) reports this method of disposal proves satisfactory, but that the supernatant liquor dries slowly. Bradley (Buchanan) disposes of supernatant liquor to a lagoon, as does Russell at Rochelle. Lovejoy (Laurens) discharges supernatant liquor to the river.

Supernatant liquor disposal constitutes one of the most common operation problems, particularly in plants practicing its disposal to the raw sewage. At plants utilizing this method of disposal, it is a helpful precaution to provide an alternate method, such as a lagoon or drying bed, to be utilized when a poor quality supernatant is present.

A poor quality supernatant liquor, if discharged to the raw sewage or to the aeration tanks, may result in a poorly settling activated sludge, which results in larger volumes of sludge for transfer to the digester, and thus in larger volumes of supernatant liquor for disposal. Whether a plant can handle supernatant liquor in the aeration tanks without pretreatment depends on the degree of loading of those units and on the quality of the supernatant liquor.

OPERATION AND MAINTENANCE OF SMALL PUMPING STATIONS*

By GRANT M. OLEWILER

Supt. Sanitary Drainage, Lower Merion Township, Pa.

Four sewage pumping stations are operated in Lower Merion Township.

Station A, the Mill Creek pumping station, is equipped with three electric-motor driven two-stage horizontal centrifugal pumps. It operates against a head of 252 ft. when using its largest pump, which discharges 1,600 gal. per min. through 6,671 ft. of 12-in. force main and at smaller heads when its other pumps, one of 1,250 and the other of 1,000 gal. per min. are in use.

Station D, the Gulley Run pumping station, is equipped with duplicate electric-motor driven horizontal two-stage centrifugal pumps. Each pump operates against a head of 175 ft. when discharging 500 gal. per min. through an 8-in. force main 4,230 ft. long.

Station C, the Cynwyd pumping station, has duplicate electric-motor driven single-stage vertical centrifugal pumps, each of which operates against a head of 95 ft. when discharging 600 gal. per min. through 3,510 ft. of 8-in. force main.

Station B, the Ardmore pumping station, has duplicate electric-motor driven single-stage horizontal centrifugal pumps, each of which operates against a head of 52 ft. when discharging 833 gal. per min through 3,310 ft. of 10-in. force main.

FROM the following descriptions it is evident that we have quite a range in sizes and although I have seen smaller stations as to head or capacity, I consider our equipment as illustrative of small pumping stations.

Pumping stations are permanent equipment and the operator or engineer in charge is urged to be ever alert to take advantage of the many new aids to operation which are constantly being found to be basically sound and well constructed.

The following divisions of the subject have been made: the pump; the motive power; auxiliary equipment; the building; personnel.

The Pump

The pump seems to be the very heart of the plant and must be considered of primary importance. It consists principally of an element to create pressure on the sewage and devices to maintain alignment of these parts.

The next time those rags get around the shaft, rubbers wind into a tight ball between the back of the impeller and the casing, a block of wood gets between the face of the impeller and the casing, with a resulting noise that makes you think the building is about to come down, or the pumping gauge shows a reduction in pressure or capacity, take a little time off and see what is happening.

What about clearances? The present-day welder in that reliable machine shop in town can do much to replace that metal which was ground off by the friction caused by fine sand in sewage. It is there—and it makes the finest abrasive possible for grinding the face of impeller blades unless of course you are one of those few operators who are blessed with the latest style non-clog closed impellers.

What about the impeller itself? Are there any visible cracks which need welding? We have had open cast-iron impeller blades crack away from the hub in a direction opposite to the thrust caused by the normal operation, so look on both sides if you wish to keep out of trouble. If the sewage is corrosive, look for pitted places which might seriously reduce the strength of the impeller or cause a loss of the design curve which is so essential for high efficiencies.

Is the impeller tight on its shaft? In one case we had all kinds of difficulty trying to keep tight an impeller on the high pressure stage. It was sent to the machinist several times before we were satisfied that it was tight. A wobbly impeller is no efficient machine and many so damage the shaft that a new one must be purchased.

How is the shaft where the packing presses against it? Those deep grooves mean that expensive packing is being wasted rapidly. If a shaft has replaceable wearing rings you are fortunate, otherwise you must balance the cost of packing and the appearance of that sloppy mess around the pump against the cost of a new shaft.

What about packing—do you use considerable? Why not talk with the salesman of the company from which it is obtained? He has many and varied installations and can probably advise a more suitable style designed to overcome just such a situation as yours.

Is the pump equipped with a water seal ring? If so, it is a valuable device for keeping sand out of the packing, but do not connect it with the potable water supply line unless proper steps have been taken to prevent back flow at all times.

Next look at the pedestal bearings. What is that material in the oil reservoir? Is it oil or a mixture of oil and sewage caused by the travel of sewage along the shaft from the packing to the oil reservoir. The latest designed pumps use slingers, which are very effective in stopping this flow. You can make duplicates of them by cutting rubber disks with a hole in the center of a diameter slightly less than that of the shaft and place them on the shaft the next time the pump is apart.

Does oil leak from around the opening through which the shaft passes? There is a rather inexpensive sealing ring made especially to prevent this.

What about bearings—are they wearing evenly or are you getting all the wear on one side? If so, it might be a good plan to try some aligning. It will pay in the reduction of power consumed as well as to make parts last longer. How did that word alignment steal into this paper? Why, that is one of the greatest troubles. We had one vertical installation which ran several years without trouble and then suddenly became a fiend. We lined this and lined that, placed steady bearings, which seemed to make it more unsteady, replaced base legs, made a new bed plate, all to no avail, several times a year that pump would go to pieces and new shafts and bearings would have to be installed. Pump men and machinists said that it was just the nature of that type and that they were constantly fixing similar pumps for other places.

One day we saw an advertisement of a flexible shaft using two universal joints, purchased one and installed it about a year ago, and—well we still have our fingers crossed. The same installation has a bearing placed in the pump casing so that it is subject to the pressure of the sewage. The original lubricant was a hard grease of the water pump type, which caused sand to become imbedded in it until it became almost a grinding collar. Black grease was recommended because of its adhesive nature, but when applied it did not have sufficient body to prevent sewage under pumping pressure from forcing itself back into the spring-feeding grease cup. Check valves were than placed in the line and operate in a very satisfactory manner. This pump operates only about thirteen minutes per run and during the time of running, sufficient grease is retained in the bearings to lubricate it so that as soon as the operating pressure is reduced the spring cup forces a new supply into place.

At the end of the pump shaft one finds a coupling. In all our present installations, except the one noted above, a rubber-bushed pin-type coupling is in use to connect the pump with the motor. These couplings had a wide flange around the outer circumference, supposedly to protect operators from the bolt heads, but some years ago the station operators found that a monkey wrench tightened on the flange would provide a good lever arm to turn the shaft when cleaning the pump. The majority of the flanges had notches broken out of them. When an 8-in. diameter coupling, with notches in its face, rotates at a speed of 1,750 or 1,175 r.p.m., it is not a nice thing to get against. In addition the notches tend to upset the dynamic balance of the shaft. These couplings have been turned down so that the flange is now very short. Cages have been placed so that the operator is not liable to touch them. Rubber bushings are replaced whenever inspection shows that the old ones have become worn.

The original installation of triplex plunger pumps at one station was never very satisfactory. The sand in the sewage constantly cut the cylinder walls and piston rings so that in a very short time the back-pressure slip became considerable and the pumping capacity would become greatly reduced. The poppet valves were a constant source of trouble and had to have their covers removed and be cleaned of rags, rubbers and other foreign material which seemed to foul them constantly.

The Motive Power

This now brings us to the motor and its bearings. While some trouble has been experienced with the bearings, it has been very slight and can be traced to poor alignment. We once walked into Station "D" and found that practically all the oil

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had splashed out of the bearing when its cover had become dislodged or was not replaced correctly. Oil escaping from the ends of a motor reservoir at this station was eliminated by sealing rings.

We have been very fortunate with electric motors, one of which has been in operation since 1919, in that they are practically free of trouble, but it has been reported that years ago the windings on one motor at Station "C" burned out during the night, probably due to bad fuse protection.

Because of the increasing age of our equipment we recently purchased a megger, volt and ammeter and expect to make periodic tests to obtain information on the condition of insulation and current consumption. Articles on electrical equipment maintenance in Sew. Wks. Jour., May, 1941, "Operators' Corner," are very helpful in themselves and for the references which they contain.

While the multiple-cylinder gasoline auxiliary engines now being installed as standby power should be very reliable for small motor-driven stations, I regret that the only time I was at an automatic station where this equipment was installed, the operator opened the main switch to demonstrate the unit and although the battery turned the starting motor several times the engine failed to start, presumably because of a low battery condition. This, more than anything I have ever seen in outside stations, impressed me with the tremendous importance of proper maintenance because of the potential danger in such a condition. Had that been a real electric power failure the plant would have been out of commission until the electric power was restored.

Auxiliary Equipment

The most important auxiliary equipment is the switchboard and starter to cut the motor into the line. Two of the motors at Station "A" are started with a manual control, by which the current is turned into the motor with grid resistance in the circuit and the resistance is then cut out by a manual trolley control. Overload switches automatically break contact should an overload be applied. The most constant source of trouble on one board is the contact fingers in an oil-bath switch. Although made of laminated spring copper the contact does not seem to be sufficient to break without burning, and arc tips must be replaced frequently. For many months the men did not understand the operation of this switch and the fingers were allowed to burn to such an extent that the entire rod had to be purchased rather than the tips, which are made separate so that they can be replaced at a small cost.

The third motor, the newest, is started by an automatic industrial controller, actuated by a float-switch control or a push-button station, which brings the motor on the line by a mechanical timer which releases the resistance at the end of a predetermined interval. In addition to the regular fuses this equipment is protected by a thermal control overload switch.

Prior to the beginning of this year all operation of this station was manual, since the pumps are horizontal type two-stage centrifugal pumps located above the level of the wet well, and it was necessary to prime them prior to starting. At that time automatic air release valves and a vacuum pump were installed to make the equipment self-priming, and to eliminate almost constant trouble with check valves. A float and control switch were connected to operate the automatic industrial controller so that we are now able to operate this station on two shifts in place of three.

Tests of the pressures found in the water seal ring in the packing stuffing box on the high pressure stage of the intermediate size pump proved that a small plunger pump, which supplied water from a reservoir connected with the adjacent stream, was not delivering sufficient pressure to overcome that in the casing, with the result that the sewage was not held away from the packing. Because the stream bed had washed out since the construction of the reservoir to such an extent that water could only be obtained by placing a row of stones slightly below the inlet holes, this method of obtaining water was abandoned and we installed a small storage tank in which the supply of city water was regulated by a float valve and ample pressure was supplied by a motor-driven centrifugal pump of small capacity manufactured especially for such a use.

An amusing incident which occurred when this pump was installed illustrates the value of observation and thought. Only one wall of the station was unencumbered with other equipment and this was selected as the best possible location for the electrical control equipment for the pressure water pump and vacuum pump for priming. The installation was made in cold weather and as soon as the connections were made a test run was made. The pump motor had hardly started before the thermal control to prevent operation of an overheated motor kicked out and the pump stopped. Examination of the motor showed that it was quite cold to the touch and that the wiring had no defects but a second run again threw out the thermal control. Here was indeed a mystery until someone observed that the switch had been placed directly above a heating radiator which at that time of year was quite warm and this heat, together with the use of a thermal unit which had the lowest possible heat release recommended for this size motor, had combined to cause the equipment to refuse to operate. The next size thermal unit was obtained and no further trouble was experienced, but I can easily imagine the fun which we would have had if the installation had been made during the summer and after operating with entire satisfaction for several months, it suddenly refused to work as soon as the weather became cold enough to cause the heating plant to be forced a little more than usual.

At Station B, where the pumps also are located above the level of the sewage, automatic priming of the pump and automatic starting have also been installed. Because of the very much smaller head against which these single-stage pumps operate, no water seal rings are placed in the stuffing box, but the packing and shaft are cooled by a water jacket through which city water is circulated. Automatic stopping and starting of the motor also required an automatic shut-off for this water line. A very convenient solution of the problem was found in a solenoid controlled water shut-off valve which starts the flow of cooling water as soon as the pump is started.

Stations C and D, being newer, have the pumps located below the level of the sewage. Starting and stopping is regulated by industrial controllers actuated by a float switch connected with a float in the wet well. That "connected with a float" sounds quite simple and yet for years we were plagued with constant trouble caused by breaking of chains and cables on floats, especially since these breaks caused the pumps to run until the operator arrived in the morning. About a year ago chains and cables were eliminated and all control floats were placed on lever arms made of galvanized iron piping. Connections to the switches were redesigned after the failure of efforts to purchase switches of the type which we needed because the manufacturers stated that they could not supply such an automatic switch. We have since learned that the Westinghouse Co. makes this equipment but are now well satisfied with our own design.

Because none of our stations were provided with venturi meters or other devices to measure the inflow and pumping, we have constructed a pitometer station on the force main from each station to test the pumping capacity, but at this writing sufficient data are not available to elaborate upon their use.

Recording pressure gauges with daily charts are installed at each station and approximate reports of pumping are compiled from these data. Additional large dial pressure gauges are placed on the force main to inform the operator of the condition of his pumps from a distance.

In an endeavor to reduce the pressure fluctuations in the force main caused by the operation of the check valves and the surge of sewage, air-cushion chambers have been installed at Stations A, C and D, but it has been very difficult to maintain air in the chamber. The only case of serious flooding of the dry well at Station D was caused by the breaking of the air cushion sight glass during the night. By morning the level of the sewage had risen to the top of the motor, which had been stopped by a piece of burlap, used as a foot wiper on the bottom step, becoming wound around the shaft until it acted as a brake and actuated the overload cut-out switch. A control coil which burned at the same time may have been caused by the splash of sewage. Fortunately the motors were waterproof and after being washed out and dried for a short period, the duplicate motor, after having the old oil drained and new oil placed in the reservoir, was started without damage. The motor which had operated during the night was taken apart when inspection showed that the force of striking the sewage had either broken or badly bent the metal fins on the rotor which act as a fan to provide air circulation within the motor. New fins were placed and after tests to determine that it had suffered no other damage this motor was also placed in service.

At Station A, an automatic surge pressure-release valve has been placed on the main to relieve excess pressure by discharging back into the wet well as soon as a predetermined pressure is exceeded.

Since power charges are based upon the total demand load which may be applied at each station, the control panels must be so arranged that no two pump motors of duplicate installations may be operated at one time. This is effected by using de-energizing coils or contacts to render inoperative the starting control of the other motor during the use of one. At station A, where three pumps are to be operated, a large knife switch is

used to determine the motor which is to be placed in service. If the handle is placed in the closed position on the right hand side the smallest pump may be operated; if closed on the left hand side the intermediate size, and if placed in the open position the largest size may be used. Again interlocking coils prevent the use of the other pump motors.

In automatic stations, where no operator is present for long periods during the day and night, it is highly desirable to be able to operate the second pump at a predetermined time after the first fails to operate. We have therefore arranged float switches so that if the level of sewage in the basin continues to rise beyond the point at which the first pump should operate, a stop will engage the switch for the second pump and because of the interlock will disconnect the first pump. This has the advantage that opening of the overload switch will remove any damage to the first board without placing the station out of order, while a temporary loss of pumping capacity in one pump will be taken up by the operation of the second pump.

The Building

All our stations have a screen and grit chamber, a collecting well and a pump house.

When non-clogging impellers came into use the bar screens were removed. Trouble with foreign materials, especially in the two-stage pumps where the shaft passes through the suction elbow, has resulted in the replacement of the screens and strict requirements that they be cleaned whenever the level of the sewage on the upper side of the screen exceeds that on the lower side by an amount which will cause backing of sewage into the inlet pipe.

Screenings are placed in a compartment with a sloping floor and a drain to the screen chamber. After draining they are placed in covered cans and hauled to the township incinerator. Prior to the use of the separate drying compartment, the incinerator operators complained about the difficulty of burning this material, but since they have been used we have had no complaints.

Examination of the material removed indicates that the majority of all the screenings could very well pass into the basin without harm to the pumps. Ours is a separate system relatively free from large, objectionable materials, but this large proportion of unobjectionable material must be removed in order to prevent rags and long, stringy material from entering the pumps and fouling them and the priming equipment. For us the answer seems to lie in some form of shredder which will reduce this long material to small pieces and eliminate the cleaning of screens. Unfortunately the stations are located so near to the road that no space is available for equipment of this nature, but we are still looking and hoping that somehow it can be fitted into our plant.

Sand chambers consist of small compartments where the velocity of the sewage is broken by baffles and a large portion of the grit content is deposited. These are cleaned by the sewer repair force whenever the operators find that they are becoming filled. Materials removed are buried in large holes dug in a meadow adjoining the Mill Creek pumping station and covered for a few years, after which time the Shade Tree Commission generally finds use for the material as a soil builder. Until recently the holes were cleaned out by local gardeners who had also discovered its value, but this practice is no longer allowed.

Collecting wells vary in size from 205,000 gallons at the Mill Creek station to 7,000 gallons at the Gulley Run station. Slope on the floors ranges from 1 to 70 at Mill Creek to 1 to 4 at Gulley Run. In spite of the seemingly flat bottom at Mill Creek, sludge beds do not form to a great degree because sewage is pumped down to the level of the sump or suction hole at each run of the pumps. The sand which escapes the grit chambers is removed at least once a year and more often if an accumulation is noticed. Greases and fats which collect on the side walls of the wells are removed by scraping about twice a year, while float balls are cleaned quite often. Improvements in the operation of the Ardmore and Mill Creek basins were effected by cutting larger openings through partition walls originally designed to direct flows but which were operated to produce eddies with resulting accumulations of sludge.

The wet well at the Gulley Run station is divided into two compartments which are connected by an opening fitted with a sliding gate valve, so that either side may be used for cleaning.

In normal operation the entire flow of sewage enters one compartment and passes into the second compartment through this valve, with the result that the larger portion of the heavy material remains in the basin which it first enters, while the second side remains relatively clean. As the impeller openings on these pumps are quite small and clog very easily, the pump connected to the second or clean side is used during the night when no operator is assigned to the station, while the other pump is used during the day when a man is available to remove stoppages.

Because two of our stations have the pumps located below the collecting well it is necessary to place them in a dry well. At the Cynwyd pumping station the pumps are mounted vertically in what amounts to a basement because its only connection with the upper floor is an "L" shaped stairway. An original heating unit, consisting of a cast-iron laundry type barrel stove, was placed on the first floor but failed to provide heat for the basement. The recent installation of a hot-water boiler and a radiator along one wall of the basement, together with ample radiation on the upper floor, has dried the walls to such an extent that no moisture is evident, and has reduced materially the amount of coal consumed during the heating season. This left several months during the year when we were still troubled by the excessive moisture in the air. An exhaust fan has been installed and with this in operation a considerable degree of relief has been evident.

The Gulley Run station has horizontal pumps placed in a dry well, but in this installation the ground floor consists only of a balcony 4 ft. wide around three sides of the room. The original heater was located on this balcony but was soon found to be unsatisfactory and was moved to the lower level, where it provided only a slight improvement. We expect in the near future to re-design this plant and install additional radiation. The normal ventilating areas in shop-type windows and a roof ventilator have failed to produce sufficient circulation of air for the summer months when no fire is in the boiler, with the result that on humid days water will actually run from the pumps and piping so that it is difficult to keep the paint looking presentable. This condition at times extends above the balcony and one summer day became so bad that although the operator removed the recording pressure gauge chart several times during the day to dry it, there is no record of pumping during the night because the paper buckled and held the recording pen away from the chart.

The two other stations have basements only large enough to contain the heaters and coal storage bins. With the exception of the Gulley Run station all basements have caused trouble because of infiltration of ground water. At the Cynwyd station we excavated around the outside of the building and placed a heavy coat of cement mortar on both the outside and inside of the concrete wall. The interior walls were plastered at the other stations.

Although the newer stations have provision for hoisting the equipment built into the walls the older ones did not, and since they contained the heavier machinery, many of our men have suffered from body strains caused by trying to lift or move heavy motors and pumps. Several years ago plans were made to install beams and traveling hoists. We came across a catalogue of a bridge-type hoist made for handling automobiles and rated at a capacity of two tons, which sold for about $125.00. Since none of our individual pieces exceeded this load we purchased one for trial. Shortly afterward we had to dismantle a pump at the Mill Creek station and in order to do this we have to move the motor and all. While no time studies were made to compare actual costs, it is our belief that on this one job alone we saved the most of the hoist in labor time. Needless to say the second hoist was purchased immediately.

The outside of the stations was painted each year or two but the paint was so thinned to make it go a long way that in less than a year the checks and blisters would become so pronounced that it would have to be done over. Painters' torches were purchased and all the old paint was removed from the woodwork by burning and scraping. The wood was given a priming coat; undercoat and enamel were placed on interior surfaces and two coats of lead and oil paint were placed on exterior surfaces. Openings between wood window frames and the brick walls were caulked with oakum and sealed with caulking compound. Steel was placed over windows and doorways where cracks had appeared in the brick walls and the cracks were cemented. The old brick walls were cleaned and painted a light buff color. Floors were cleaned and painted a light gray while pumps, motors and piping were painted black. It does not take long to write it, but many a weary hour was spent by the pumpmen in scouring floors and walls to remove oil and grease which had accumulated for years.

The job has been a big one and although all the spare time of the men has been devoted to it, and by the time it is finished two years will have elapsed since the work was started, but we now have a sound job which will last several years before it will have to be retouched.

Serious and amusing incidents have happened during the work. At one time, acting upon the advice of our paint supply house,

we had sealed the concrete wall in the Gulley Run station dry well with water glass and then applied a coat of recommended paint. The next morning, when the operator entered the plant, he found that the paint had run from the walls and was on the floor. A call to the supply house brought the manager, but since he had never seen anything like it another call brought the chemist from the paint company. He soon saw the trouble, called our attention to a fungus growth on the wall, explained the difficulty in killing it but advised that we try washing the wall with copper sulfate and painting it before the growth could get a foothold. This was during the summer when moisture conditions were bad, so we waited until the next dry day, did as directed, and although the wall has been painted for a year, we have seen only slight traces of regrowth, which we expect to kill with another application of copper sulfate.

Personnel

Schedule of Working Time.—The force operating the stations consists of the assistant superintendent of sanitary drainage and nine pumpmen. Until recently these men had to cover three eight-hour shifts at the Mill Creek station (12-8; 8-4; 4-12), two at the Ardmore station (8-5, day and night), one each at the Cynwyd and Gully Run stations (8-5, day), a total of seven shifts. Each pumpman gets two days and one Saturday and Sunday off per month as well as two weeks' vacation and whatever holidays can be arranged.

In order to give the men advance notice of their Saturday, Sunday and holidays off as well as to show a roughly equal assignment to night and day work at all the stations, an annual schedule is prepared and posted at each station. With seven men required for daily shifts two men are available to work as relief for men off or sick and to cover shifts where the men are on vacations. The most versatile man, a carpenter by trade, is used whenever available from relief work to help with maintenance, while the other relief post is moved among the men at three-month intervals.

Although it is not encouraged, the men are permitted to exchange shifts if they show a justifiable cause, but since the schedule is arranged to have all Saturday, Sunday and holidays off during daytime shifts, the men who exchange shifts must surrender their holidays off during the time that they take a night shift from another man.

This requirement is caused by the loss of the relief man for two days each time he substitutes on a night shift.

In addition to the annual schedule a daily schedule is prepared for each half month showing where each man is each day so that there can be no chance of a misunderstanding.

As this scheduling of the nine men in seven positions could make no allowance for sickness, it is necessary in emergencies to have one man assigned to look after both Cynwyd and Gulley Run stations, where operation was controlled by float switches. Since the men must supply their own transportation the operator would first stop at the station nearer to his home, see that it was operating correctly, do his housekeeping and then go to the more distant station until early afternoon, at which time he would return to the first station until time to leave for the day.

The three shifts at the Mill Creek station were made necessary by the failure of the basin to hold the inflow between the hours of 5 and 8 p.m., but, since the installation and satisfactory operation of the vacuum priming and float switch control, the schedule at this station has been reduced to a day and night shift (8-5). This has eased somewhat the pressure for men and has released another man for general repairs and maintenance for a portion of the time.

Schedule of Operation.—Prior to the adoption of the present organization the men were instructed as to their general duties and then left on their own with a daily visit by the engineer in charge of pumping stations.

When the supervision of the pumping stations was given to the assistant superintendent he prepared a definite program of work which must be done each day. As this varied in minor details at each of the stations, typewritten instructions were posted on each bulletin board and the operator for each shift of the two-week period is held responsible for the fulfillment of his share of the work.

It seems that each operator had a different idea on how the fires to heat the building should be tended and in most cases these ideas violated many of the accepted standards recommended by the heating engineers. Check dampers were not used, fire doors were left open instead, fly ash and soot were not cleaned from the top of the hot water sections and often accumulated to an extent that it was remarkable that the heater gave out any heat. Standard directions were obtained from a local coal dealer, posted with the duties of the operator and the men were informed that the fires must be controlled in this manner. Conditions have improved to a marked degree but it is pretty hard to change a man's system after he has used it forty years.

The men have entered into the spirit of the schedule and after a station has been painted and placed in good condition they are anxious to keep it looking its best.

Selection.—The present operators were drawn from seven trades—three carpenters, one gardener, one taxi driver, one cement finisher, one machinist, one auto mechanic, one electrician. Length of service ranges from 29 years to 7 months with an average service of 14 years.

With the exception of the most recent appointment the men were selected largely upon the recommendation of the commissioner of the district from which the man to be replaced had resided and appointments were based chiefly upon the fact that the new man had a good reputation in the community and needed a job. Anyone could learn to start and stop the pumps, clean the station and keep it warm!

In the case of the last appointment, at the request of the department, consideration was given only to men who had experience with motors, lining shafting, with switchboard work or as a millwright.

The older men are very willing to follow instructions and to help in any way possible but cannot be expected to advance ideas to better operation when they are not familiar with the basic theory. It is true that a large portion of the job is that of a janitor, but the value of the men to the municipality lies in their ability to operate the stations properly, the smallest of which will cost over $10,000, and not only to keep them in condition but to better the operation by improvements as they are developed by the engineers and manufacturers.

To the representatives of the municipalities I want to say—it is not good economy to turn your plant over to that aged policeman or road foreman. He is a good man and needs a job, but poor operation or inexperience will soon cause your equipment to cost you more than the difference between his salary and that of an experienced man.

Designing engineers, you have done your best to create a good plant, but your job is not complete until you place it in competent hands. It is your duty to impress the municipal authorities concerning the vital necessity of good operation.

Operators, it is up to you! Unless you keep your station always at its best, study your equipment to otbain a maximum output for a minimum cost, keep an eye open for modern improvements, learn how and why each unit acts and then show that you know this by timely suggestions or recommendations, you cannot expect to be recognized when that most important day of the year arrives—the day when the annual budget is considered.

Discussion

By J. R. Hoffert
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To give added weight to Mr. Olewiler's comments, let us emphasize the importance of periodic checking of the overall plant efficiency. Payment for power is a principal cost in the operation of a pumping station and it is nearly as continuing as fixed charges. Rising costs for power will probably be quickly noticed by the accounting force—if you have one and it is alert—and yet it may be surprising how long it takes at some plants to discover reduced efficiency, which may be ascribed to increased sewage flows, increased friction in force main, poor power regulation, or unfortunate "demand" charges.

Moving parts do have an unfortunate habit of wearing and increased clearances of wearing rings, a slipping impeller or a broken or bent one, electrical losses, or a variety of defects may markedly reduce operating efficiencies and correspondingly increase costs. And even the best non-clogging pumps do not have a sufficiently high efficiency to waste any of it.

So determine your initial plant efficiency as soon as your plant is well tuned up and thereafter keep that as a goal. When costs rise, track them down by a process of test and elimination until you have localized the trouble. The pocketbook nerve is particularly sensitive.

But checking efficiencies requires that you measure your discharge and this brings up the importance of providing some sort of reliable meter in the original construction of every pumping station. Lacking a meter, preferably of the recording venturi tube, Kennison, nozzle, Parshall flume, or similar non-clogging type, it may be necessary to resort to some type of weir, to a pilot tube traverse of the discharge line, to measuring the rise in the chamber receiving the pump discharge, or even to noting the time required to pump down a sump and estimating the amount of inflow. Actual total head at the center line of the pump, including dynamic suction head, must be determined. This is most conveniently measured by means of a large diameter, carefully calibrated bourdon pressure gage on the discharge line and a similar vacuum gage on the suction line. Power

consumption is best determined by the proper electric instruments but if these are not available, a good approximation can be had by counting the revolutions of the disc of the station meter for a sufficiently long period (with other power-consuming devices temporarily cut off) and applying the proper meter coefficient, which will either be marked on the meter or can be obtained from the power company.

A book might be written upon the testing of efficiencies but a simple method for computing efficiency is given by Homer Beckwith in the 1939 Journal of the Pennsylvania Water Works Operators' Association and in the May, 1940, "Year Book" issue of *Water Works and Sewage*.

Correcting unnecessary consumption of power will pay big dividends and so will the elimination of unnecessarily high base charges. It will seldom be possible to pump principally on "off peak" power but reducing "demand" charges by installing only that rated horsepower actually required; by interlocking motors so that only certain motors can be operated simultaneously; by avoiding unnecessary use of auxiliary power when the main motors are running; and by careful regard for good power factor, will quickly repay any effort on this work.

Good pump packing practice; exclusion of water, sewage, and grit from bearings; proper alignment of parts; a careful choice of lubricants; and a regular schedule for oiling and greasing and renewing oil—all these and many more routine tasks are worth emphasizing again and again. Particularly would we stress the desirability of having a regular scheduled time for each item of maintenance and a card record of it so that no item is not at some time overlooked.

Rigidly supported but self-aligning bearings for long, vertical pump shafts might have been mentioned and reference could have been made to the present practice of using heavy H-beams with their top half filled with concrete, for support of such bearings.

Self-priming pumps do save many a headache for the operator whose pumps have a negative head or "lift" on the suction side. Air cushions which actually keep air out of the force main and pressure relief arrangements to reduce surge pressures in the force main; dependable arrangements to insure that the standby unit will function if the operating unit fails; auxiliary power arrangements, and emergency overflows may prevent what would otherwise be a serious situation.

Some protection against clogging is necessary for all but the larger non-clogging pumps, but screening cutters of the comminutor or triturator types will serve the same purpose and at the same time overcome one of the most vexatious tasks of the operator—the disposal of screenings.

Neatness of plant and grounds is essential—it marks a good operator and helps sustain his morale. Judicious use of paint and cleaning compounds; polished "brightwork" or clean, painted surfaces; rubber floor runners, and well kept switch gear and controls speak for themselves, and attractive grounds appeal to every visitor. Leaking basement walls can cause no end of annoyance and ruin appearances and equipment. However, much can be done to remedy such conditions, even against outside water pressure, by using the best of the fast setting cements or certain waterproofing materials.

But constant maintenance is the crux of good pumping station operation whether it be switchpoints, fuse protection, automatic float control, automatic cutovers or just plain cleanliness. And it can't be emphasized too much that no station is so completely automatic that it does not require systematic maintenance and daily supervision to keep it "automatic."

Which brings us to that most important, most delicate, and at times temperamental part of the pumping station—its operating personnel. Regardless of how hard it may be to attain—even in your state—the goal should be to employ only competent, honest, dependable and versatile men, even at the expense of some "political" disappointments—and then to direct them with tact, fairness and decision.

Mr. Olewiler has had to accept his sewage pumping stations as they came to him with such modifications as time, funds and experience permitted. I would like to carry the operation of sewage pumping stations further back—to the time when the station is only some lines on a drafting board or some thoughts in the mind of the designing engineer—for the design and construction of the station have such a vital effect upon the operation of the station.

Pumping stations—even small ones—are of such varied types and include so many devices, that a volume could easily be written upon the subject. They may be isolated or part of some other building, such as a control house; they may be simple or complicated; completely automatic or manually operated; in nearly continuous operation or operating only intermittently; may use gas, diesel, steam or electrical power; may be large or small factory-fabricated units placed wholly underground; and may vary widely in other respects.

But essentially, so far as effect on operation is concerned, they include the tributary main collecting sewer, a sump of some kind; the room containing the pumps or at least the prime movers, controls, etc.; and the main force.

It may be carrying coals to Newcastle to mention some very obvious things which you gentlemen well know, but I have reviewed many pumping stations and designed some others, and it is surprising at times how often the obvious is overlooked. First, then, a few general remarks about the general design of the station:

It need scarcely be remarked that the site should be appropriate. It should be low enough to receive the sewage from all the area which will be tributary within a reasonable time in the future, with costs of lengthened main collecting sewer and force main and higher pumping head for a lower location balanced against the most of prematurely moving the station farther down the drainage area for a station at a higher location. This need not unduly penalize costs. Smaller pumps may be installed at first, or additional units may be added in the future, or a smaller impeller may be used at first, with some sacrifice in efficiency.

The character of the neighborhood will have a vital bearing upon the station. The station may have to be and can be located in the midst of a closely built up, high grade development, but this usually isn't desirable. But wherever located, the station should be in scale with the surroundings.

It should be architecturally pleasing in appearance. This does not necessarily mean it must be costly, since good appearance is largely a matter of the careful balancing of wall masses and openings and a choice of lines.

The grounds, however large or small, should be attractively planted with trees and shrubs appropriate to the site and of such variety of flowering seasons and habit as will maintain a good appearance for the longest time with the least labor and the least damage from dogs, insect pests and adverse weather conditions.

The station should command attention by its appearance; not attract it by its odor. If you have a very long collecting sewer, have to hold the sewage for too long a period in hot weather, or have poorly designed sumps, perhaps it would be well to consider chlorination at strategic points to control the odors which you must not permit to occur.

Some screening protection of all but large pumps is necessary if the station is to be really automatic. Shredding devices for reducing the clogging solids in sewage can do away with one of the most troublesome features of pumping plant operation. But many plants will continue to use bar screens. Be sure these screens have sufficient area to handle the sewage between inspections; that they aren't so placed that the flow of sewage forces the screenings through them; provide in your design for a proper draining platform and at least sufficient room for the operator's feet when he's cleaning the screen; and a self-draining screening receptacle. Make the screen chamber or pit decently accessible and provide means to lift out the screenings can. Don't ask any self-respecting operator to lug a G.I. can of wet screenings up a long set of ladder rungs and then shove the can through a glorified manhole. Neither is it necessary to provide a ship's crane to remove the screenings. An overflow channel to bypass the screen is a very useful device. And an emergency sump overflow—used for emergency only and where there is no real menace to health—can be a very ready help in time of trouble—even if it must be sealed by the state health department—and may prevent what might otherwise be a very serious situation.

Wherever practicable, divide the sewage sump in two parts interconnected by gates of some sort—it certainly will be useful in case of repairs or cleaning. Give adequate slope to the floor of the sump to prevent stranding of solids and terminate the pump suctions in a low point of the sump in such a manner that the pumps will not lose their suction until the sump is practically empty. Also, provide convenient facilities for flushing the sump and screen chamber. And put the float controls where they won't be undesirably affected by pump action.

Let the building be sufficiently ample in size to make all the equipment conveniently accessible. It may cost a little more in the beginning but it pays large operating dividends. If an operator has to crawl over equipment while watching that his coat tail doesn't catch in some rotating part or that he doesn't make some electrical contact; or if he has to solve a problem in geometry each time that he tears down a pump or breaks a pipeline—well, maybe he just won't do it soon enough the next time.

Pump controls and switch gear conveniently arranged around

or near the walls of the room with the pumps and motors out in the room with ample space around them, and piping in trenches or so disposed above the floor as not to be a trap for the unwary, do much to help operation and improve appearance.

And let there be at least a washbowl and in larger stations a toilet and perhaps a shower and locker room for the operater. And at least a small shelf or desk for making out reports, keeping records, etc. A suitable cupboard to keep equipment which might otherwise litter up a corner, and a well kept tool board will make for convenience and good appearance. Ample window area for light and ventilation should be provided. Steel sash is added protection against vandalism.

We just mentioned ventilation but it is surprising how many otherwise well designed pumping stations are so deficient in good ventilation. The penalty is either stuffiness and possible danger from gases, or, more generally, sweating walls, dripping, rusting equipment, scaling paint and wet floors.

Wherever possible there should be complete separation between the wet and dry wells or the operating floor. A roof ventilator with a damper is desirable but provision should be made to insure a good circulation of air in the dry well and in all inside rooms or compartments and to insure venting of the wet well.

In some cases all that is necessary is to carry a duct to a point near the floor of the dry well and to rely on stairway openings or floor gratings to provide circulation. One large power company has very successfully ventilated its underground structures by merely carrying two small pipes from the same level in the spit to a point some distance up the side of an adjacent outside pole with the one pipe carried a little higher than the other. This slight difference provides a continuous gravity circulation of air which keens the pits dry. However, positive ventilation by fans is so simple and so inexpensive that it is increasingly the best answer. But by whatever means, get good ventilation.

Also, provide a sump pump or at least a small valved line on the suction of the main pumps and carry it to a shallow sump in the floor so that water on the floor may be effectively removed.

Thus far we have said very little about what you may consider the heart of the whole station—the pumps and their motors. Well, since the designer does consider them so important, they usually do receive much careful consideration. And they cover such a very wide field that they are a subject in themselves. Even so, there may be some room for comments about them here.

The pumps themselves are nowadays nearly always centrifugals. Fortunately for the operator, the open shaft vertical pump in the wet well is not used so much any more. Even the enclosed shaft motor in the wet well offers him enough trouble.

A dry well increases the size of the station but it certainly is a boom to the operator and wherever practicable should be provided, whether the pumps are vertical or horizontal.

Quick opening handholes for cleaning pumps, replaceable shaft sleeves and wearing rings, water deflectors, pressure water seals, drip piping, direct connection through flexible couplings to the prime movers—are all highly desirable. Shafting should be as short as possible.

The pumping capacity should be adequate and in duplicate for all normal requirements. If pumping is to be direct to a sewage treatment works, there will be less disturbance to the treatment processes if the pumping can be done by increments in pump capacity and sufficient sump capacity can be provided to permit the pumping to somewhat approximate sewage flows or help smooth out sudden peaks. This should, however, not induce too ready adoption of unduly small step changes by variable speed motors which carry their own troubles.

Wherever possible, the pumps should be so placed that they will have a positive head upon their suction lines and thus be self-priming, otherwise an automatic priming device should be provided so as to do away with troublesome foot valves.

In order that each pump shall carry its own share of the load, each should be provided with a suction line direct to the sump but these should be interconnected by a manifold properly valved. The discharge piping should be carefully designed to meet its hydraulic requirements.

Again we would emphasize the desirability of providing, at the time the plant is constructed, a suitable means for measuring the discharge of the pumps.

Adequate standby power is essential in direct proportion to the seriousness of power failure. Connections to more than one public utility, an emergency generator, gasoline power, or whatever is most practicable and necessary, should be provided.

It would be possible to consider so many other features but this discussion has already exceeded its proper proportions—like the assisting minister who preaches his sermon when offering the prayer. But we cannot close without one final word. Don't make, permit, or continue any connection between a potable water supply and any sewage handling device or pipe lines. If a float tube or meter is to be kept free of sewage solids by means of a small flow of clean water, see that it enters it from overhead by gravity through an air gap. If a venturi ring or some other device is to be flushed out by water pressure, see that the connection is only a temporary one which must be held in position by the operator while using it—and then don't permit it to be wired or clamped permanently in position. Be sure water seals are supplied from tanks supplied in turn by water entering over the top of the tank through an air gap, or use a water seal pump taking its supply from such a tank. Remember there's only one safe rule—don't permit any cross connection. The wholly improbable or the "impossible" has actually happened too often to take any chances.

PACKING POINTERS

The following suggestions and warnings, based on the many years of practical experience of a prominent eastern packing manufacturer, will be found valuable in the installation of packings. To follow them may mean freedom from trouble that would occur when even the best packing is improperly installed.

1. NEVER fail to make clear to the manufacturer what the packing is to pack. In other words, with what will the packing make contact?
2. NEVER guess at a size, diameter, width, or height, when ordering packing. Many failures result from small errors in dimensions.
3. NEVER allow too little or too much clearance between follower plate and inside cup wall, or any metal parts behind any upright leather wall. Clearances of 1/16" to 3/32" on each side, depending on size of packing, is recommended.
4. NEVER doctor packing lips with a pocket knife or file, or batter lip in installing. A rounded lip destroys the seal. Packings can be softened by soaking in warm water if necessary.
5. NEVER apply cup packings with flange too high for clearance at the head of the cylinder. Keep systems clean, so sediment will not collect in cylinders and wear the packing.
6. NEVER apply cup packings to end of badly worn plungers; pressure will force the packing down into clearance between plunger and cylinder. Leather washers cut from old cup packings can be made to reline such equipment to proper dimensions.
7. NEVER apply "U" packings where plungers are worn without reinforcing under the heel with leather cushion rings.
8. NEVER look for long service on scored plungers or in scored cylinders; resurface or reline such equipment to proper dimensions.
9. NEVER allow water to corrode cylinder walls, plungers, stems or rods. Use an anti-rust agent that will prevent this and will act as a lubricant.
10. NEVER squeeze hemp or fabric filled "U" packings in glands where such packings are to replace solid section compression packings; set gland up to point where "U" packings are free to function properly.

PROBLEMS IN HANDLING SEWAGE SLUDGE

By CHARLES R. VELZY
Cons. Engineer, New York City

THE disposal of sludge has always been one of the major problems of sewage treatment. It is not in itself a primary objective of treatment. It adds materially to construction and operating costs, and the preparation of the solids for final disposal is fraught with many difficulties. It has been a necessary evil, rather than an asset.

Sludge handling problems differ according to the type of sludge, and the magnitude of the difficulties varies as widely as the characteristics of the material being handled. Activated sludge is liquid and homogenous, and presents few mechanical or hydraulic problems. Primary sludge, on the other hand, is of lower moisture content, contains solids which are coarser and partly of a fibrous nature, and usually carries grease in an amount to be troublesome. Its characteristics cover a range from a relatively freely flowing heavy liquid to a semi-solid which refuses to move under the action of a positive displacement pump and the coaxing of a skillful operator. A mixture of activated and primary sludge may show characteristics anywhere between these extremes. This discussion relates to the problems encountered in handling sludge having the more stubborn characteristics.

At least two of the "gremlins" plaguing the sludge operator can be identified and are well known. They are the trash or ground screenings which come through the settling tanks, and the grease which is usually handled with the sludge. The troubles caused by these materials have led to the suggestion that they should not be returned to the sludge when once removed from the sewage. It is often found, however, that disposal with the sludge is less troublesome than separate disposal and the problems related to and aggravated by these materials are therefore frequently encountered.

Trouble from screenings may be reduced by attention to a number of details in plant design and operation. Screenings may be designed with relatively close bar spacing (¾ inch or 1 inch), and the grinder may be arranged to discharge the ground screenings on the upstream side of the screen. Some of the coarser material passing the grinder is thus caught again and reground. It is obvious that the grinder should be kept in good condition, particularly to minimize the passing of long fibrous material.

In a treatment works where large quantities of leaves and other trash are likely to accompany or follow a storm, it may be advantageous to provide for by-passing the grinder to keep the screenings out of settling tanks at times of heavy load. The elimination of what might constitute a "slug" of clogging material is worth while, and since the screenings at such times are relatively clean, separate disposal is not too difficult.

Grease is difficult to handle wherever it is found. Skimmings from a settling tank require mixing with large quantities of water. The water must be largely separated for economical pumping, but if separation is too complete the grease "cakes" and cannot be pumped. It tends to congeal in pipe lines, and in a sludge tank it floats and adds to the burden of control of top sludge or scum.

These problems are being minimized at some plants during the war by salvaging the grease. The skimmings from the primary settling tanks are removed and sold. Operators at these plants report appreciable reduction in grease troubles in pipe lines and in quantity of floating material in sludge tanks.

For the sake of discussion, the problems in handling sludge from the time it leaves the settling tanks until it is ready to leave the sludge tanks are classified into four categories, as follows: (1) pumping problems, (2) troubles encountered in pipe lines, (3) metering, and (4) problems in the sludge tanks.

Pumping Problems

Discussion of sludge pumping always leads to consideration of types of sludge pumps. The competition between the displacement pump and the "screw-feed" centrifugal pump will probably continue for some time. In discussing their relative merits, one plant superintendent once said that he thought an operator having both types available would always use the centrifugal so long as it was effective, because it was simple and clean to operate and required a minimum of maintenance. For high suction lift or for discharge pressure above normal, however, the displacement pump is better adapted. In a plant recently built, where two sludge pumps were needed, one of each was specified in order to cover a wide range of sludge characteristics and also to obtain the advantages of both types of pump.

It has been stated that a displacement pump operates better with some suction lift than with pressure on the suction side. However, the need for minimum length of pipe and the fewest possible bends in the suction lines often points to placing the pumps below the water level of settling tanks. Some pressure in the line also is an aid in keeping the line clear of gas. The designer must weigh these and other factors in each plant design and adopt a layout best fitting the conditions.

Considerable uncertainty is involved in determining the operating pressure for which sludge pumps should be designed. It is a problem which does not conform to hydraulic formulas, and the answer must, therefore, be derived from experience and judgment. Particularly in larger plants, pressures sometimes build up rapidly or a stoppage results in a sudden increase in pressure. A pump which will withstand and operate under a pressure of 50 pounds per square inch will suffer much less interruption than one designed for only 50 feet of head. At New York City it has been the practice to install in each primary sludge pumping station a high pressure pump for emergency use, which is capable of operation at 100 pounds per square inch.

At Buffalo the primary sludge pumps specified and built for a working pressure of 50 feet proved to be too light. Breakage occurred in connecting rods, packing gland bolts and in pump body castings. The pumps have been modified, the alterations including new connecting rods, smaller diameter cylinders and plungers, heavier castings and some other changes. Less interruption to operation has been reported.

The variable pressure, wide range in sludge characteristics, and the general severity of the service have led to more sturdy pump design. An example of later design is the triplex plunger pump with variable speed gear drive. (See Fig. 1.) It has a number of advantages over the duplex plunger pump with variable stroke. Added capacity is obtained without excessive velocity through valves, full stroke at low pumping rates results in less slippage than shortening the stroke, and there is always a more uniform velocity in the discharge pipe.

The summation of these factors calls for a pump of more rugged construction and better and more expensive mechanical parts than was acceptable some years ago. Manufacturers are meeting this requirement, and designing engineers should adjust specifications as necessary to call for the better type of pump where it is needed.

The maintenance on sludge pumps must be expected to be high. Wear on packing and plungers is rapid, and frequent renewal of packing is necessary. For plunger pumps, most operators experiment with several types of packing until one is found that lasts longer than other types. It is difficult to select any make or style of packing as best for all applications, but it is suggested that resilience and resistance to abrasion are two characteristics to be sought.

Relatively frequent restoration of plunger surface is also required. This can be accomplished by installing new plungers. It may be worth while using a plunger with a thick wall so that refinishing can be done one or more times before discarding the plunger. The added clearance resulting from refinishing can be taken up with thicker packing or a shim in the outside of the packing space.

Every operator is familiar with the problem of maintaining cleanliness in a sludge pumping station, particularly where plunger pumps are used. Leaking packing, drawing of samples and cleaning of valves, all contribute to the difficulty of keeping the station clean. Pump bases are usually provided with a lip or curb and a drain to carry off the leakage from the pump, but with sludge this is not effective. Solids from the sludge tend to accumulate within the curb and to clog the drain. Since cleaning must be a continuous and a routine operation of flushing and wiping, it may be helpful to fill the drainage spaces and channels in the pump base to prevent accumulation of dirt.

At the joint meeting sewage treatment plant of Essex and Union Counties, New Jersey, each sludge pump is enclosed within a concrete curb. As explained by E. P. Decher, acting chief engineer, this keeps the spilling of sludge "within bounds" and makes it possible to flush around the pump after cleaning valves and the like, without splashing the rest of the station floor.

In addition to thorough cleaning, Mr. Decher offers the suggestion of burning a small amount of sandal wood to overcome any lurking odors of sludge. A recent visit to the plant demonstrated the effectiveness of this "stunt." It should, of course, be borne in mind that the burning of incense is not a substitute for cleanliness, and the immaculate condition of the joint meeting plant shows that there is no improper use there. This may

be worth remembering as a final touch to the preparations for the visit by the Women's Club or the mayor and his committee.

Piping

There has been some difference of opinion regarding the selection of pipe sizes for sludge. One line of reasoning calls for large sizes and low velocity in order to keep friction losses to a minimum, particularly when the sludge is heavy. The other reasoning calls for relatively small lines and higher velocity in an effort to prevent deposits and the congealing of grease on the walls of the pipe. With the differences in sludge characteristics and other conditions in the various plants, it is difficult to make a definite determination on this point.

In recent years operators have reported a number of cases in which grease has been found to accumulate on the walls of the pipe to greatly increase friction losses, or actually to produce stoppage in the pipe lines. In many of these cases pressures have not been excessive except as the grease has accumulated in thickness sufficient to greatly reduce effective diameter. It therefore appears that a reasonable velocity in the sludge line will not cause excessive pressure, unless the line is partly plugged. When a line is partially plugged it is best that it be cleaned and normal operating conditions thus restored.

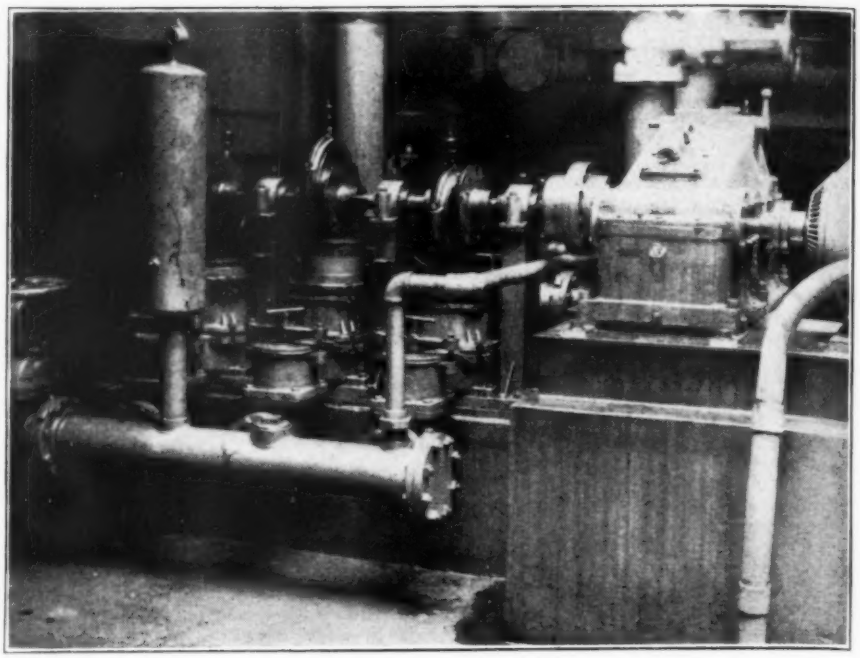

Fig. 1—Triplex Plunger Pump with Variable Speed Gear Drive

A part, if not all, of the trouble of grease accumulation in sludge lines appears to result from the pumping of primary settling tank scum through these lines. In a number of plants an operating rule has been established to the effect that scum should not be pumped alone, but should always be mixed with sludge, and that any pumping period should be ended with an appreciable time of pumping sludge alone in order that a minimum amount of scum will be left in the line to congeal. This practice has been found to be helpful.

At Buffalo a 10-inch cast iron discharge line for raw primary sludge some 600 ft. long was found to fill with grease in a period of a few months. Cleaning was accomplished with live steam introduced through an injector at the pump end of the line. The injector was made to mix water and steam and produced a flow of hot water through the sludge pipe. The grease accumulation would begin to break away from the walls of the pipe at a temperature of approximately 150 deg. F., and the pipe was found to be well cleaned at a temperature of approximately 180 deg. F. Cleaning with steam has become routine, being done approximately every four months.

The procedure is accompanied by some difficulties. In the first application of steam a section of the cast iron pipe was cracked by too rapid rise in temperature. With cast iron pipe it is necessary to both heat and cool the pipe slowly to avoid breakage. Another problem with bell and spigot pipe is the tendency for the joints to open and leak as a result of the contraction and expansion.

At some other plants, grease in sludge lines has been removed by pressure. The use of high pressure pumps at New York City plants has been mentioned. Sometimes water under high pressure has been used. It is reported that in some cases grease and other accumulated solids have been forced out of pipes in long continuous "slugs" by the application of pressure.

While these two methods, and probably others, have been effective in cleaning sludge lines, it is still advisable to make the line accessible. Outside the pump rooms the pipe may well be placed in tunnels. If this is not possible, manholes may be built around the bends, and cleanouts provided at bends and at other points for rodding and other cleaning methods, to be used if necessary.

It is suggested that, in larger sewage treatment works, sludge lines be laid in duplicate and be made of steel. This will permit cleaning the lines of grease accumulations by the application of steam or hot water and, if the duplicate lines are cross connected, this cleaning operation can be done without interrupting sludge pumping.

Metering

The metering of sludge is admittedly difficult. Although far from perfect, the Venturi meter is generally found to be the most satisfactory. Accuracy cannot be expected, and refinements in the meter design therefore are not justified. It has been found that one or two openings directly to the interior of the meter are more satisfactory than a Piezometer ring, as the ring is easily plugged with solids. Special cleaning and flushing devices and connections are also helpful.

Sometimes the use of any kind of meter for sludge is questioned because of the difficulty of obtaining satisfactory operation. It is suggested, however, that a meter is worth while, not only because it gives some measure of the rate and quantity of sludge flow, but also because it can serve as a useful indicator to the pump operator. The flow of sludge through a sludge pump may be stopped at any time through clogging of a line or lodging of some obstacle in a valve, and that this condition may not be observed by the operator without some form of indicator. A sharp drop in rate shown by the meter indicates such an interruption in flow.

Likewise, an operator may start a pump without knowing whether or not the sludge actually starts to flow. A normal indication by the meter when starting a pump is reasonable assurance that sludge is flowing, while failure of the meter to respond calls for investigation of the pump or motor, or both. It is the writer's experience that a sludge pump operator gets the "feel" of the situation so that he becomes skillful in using the meter as an indicator of pump operation, and may also use the pump to indicate the meter operation. Thus if a meter shows no flow while the pump is running, an operator may clean the valves and otherwise assure himself that sludge will flow when the pump is in operation. If the meter still does not register, it is reasonable for him to conclude that the meter needs attention.

This kind of operation appears to be troublesome and a procedure which needs improvement. There are, however, many troublesome factors in handling sludge and this is just another case in which certain maintenance operations must be accepted as routine. In operating a plunger sludge pump, it is routine to clean valves, and constant attention to a sludge meter is in the same category. Flushing of meter connections may well be done daily or oftener to prevent more serious difficulties, and further cleaning of the meter may be required at frequent intervals. In this connection the flushing system for the meter should be of adequate pressure so that the excessive pressures which sometimes occur in the sludge lines will not overcome the pressure of the flushing system.

Problems in the Sludge Tank

Probably every sludge tank accumulates scum at the top. Some prefer to call it top sludge. Whatever its name, it is obvious that some of the solids introduced into a sludge tank go to the top while others settle to the bottom. Evidence of scum accumulation is found in the answers to inquiries sent to plant operators

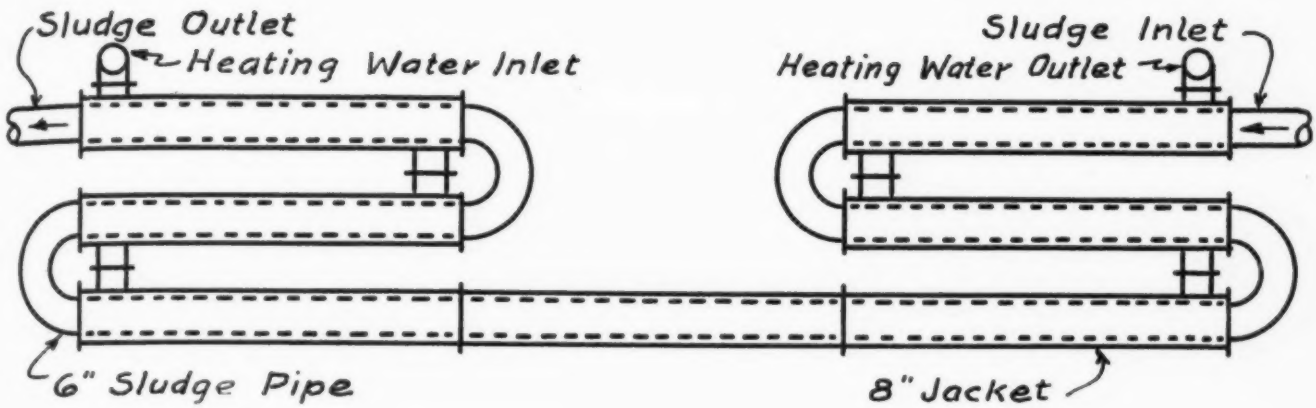

Fig. 2—Heat Exchanger for Sludge

by Wisely and reported in *Sewage Works Journal*.[1] Considerable progress has been made in designing tanks and equipment and in providing means for reducing the quantity of top sludge and for minimizing the difficulties resulting from its accumulation. It appears reasonable to assume, however, that where there is sludge there will be scum or floating solids, and that this should be recognized in the design and the operation of sewage treatment works.

The characteristics of the top sludge vary with the nature of the raw sludge and the conditions under which the sludge tank operates. They have a marked bearing on what can be done and what should be done to control or reduce the quantity. In sludge tanks of large capacity the digestion of the top sludge may keep pace with its accumulation and no problems occur, while with tanks of more limited capacity the floating material sometimes builds up to a thickness of several feet. If the solids content increases to above 20 per cent, the material will not flow in a pipe and is therefore difficult to move. It tends to become inactive and may seriously encroach on the digestion capacity of the sludge tank.

In the design of sludge tanks it appears wise to consider the possibility of top sludge accumulating to an extent which may cause trouble. Structures and equipment which keep the sludge moist have merit, and circulation is helpful. Some means for increasing moisture content and withdrawing the material may be appreciated by an operator who finds his sludge tanks handicapped with a thick layer of dense top sludge.

One arrangement for withdrawing this material which has been found effective is a pipe line carried to a point near the top and center of the tank and connected to the suction of plunger pumps. A vertical drop of some 20 feet in this pipe from the inlet point near the top of the tank to the level of the pump provides a "reservoir" into which the scum can flow as it is softened and from which the pump can draw the material. A spray in the gas dome and over the open end of this vertical pipe softens the scum until it will flow. It has been found that scum as high as 40 per cent solids can be moved with this arrangement, and that even the densest scum will gradually flow to the center of the top of the tank as the material is removed.

To attempt to make specific suggestion for operators on methods for solving scum problems is of questionable value, because no two situations are alike. Some ingenuity is required at each plant and operators have shown a high degree of skill in solving their own problems. However, a brief summary of ideas may be helpful.

Obviously, sufficient observations should be made to determine whether solids are accumulating to such a depth as to be troublesome. If that appears to be the case, measures may be applied to promote the digestion of the material. These may include increasing moisture content, circulation or stirring, and the addition of chemicals for pH control or to otherwise promote digestion.

When the situation is not brought under control by these measures, it may be advisable to remove some or all of the scum. It must be admitted that removal is not a final answer, as disposal after removal may be no small problem. Where incineration is provided, the material may be filtered and burned, with care being taken to control temperature in the furnaces. Drying on sand beds and disposal as fill is another possible method of disposal. Some effort in keeping the scum under control is worth while, as the emptying of a sludge tank is a major operation, particularly if it becomes necessary because of the accumulation of scum.

The heating of sludge tanks by hot water coils within the tanks has been common practice since the first use of separate digestion tanks. Although simple in design and operation, this method has not been entirely free of trouble. If the water is too hot, collection of solids on the outside of the coils impairs the transfer of heat. The pipes and their supports sometimes tend to retard the free flow of the sludge. Expansion and contraction of coils in large tanks requires consideration in design. Coils sometimes break and the emptying of a tank for repair of the coils becomes necessary. Mr. W. E. Gerdel reported a break in a heating coil at Cleveland in 1940,[2] and J. W. Johnson has described repairing broken coils at Buffalo in 1943.[3]

The direct application of steam has been used successfully as reported by Mr. A. M. Rawn.[4] This method is worthy of consideration where steam is available. It does not apply so well where gas is used in engines, because the waste heat from engine cooling water and exhaust is largely recovered at low temperature and must therefore be used as hot water.

The heating of sludge through some form of heat exchanger outside the sludge tank has been tried with only partial success. Heating in an open tank may produce objectionable odors. An exchanger involving passing sludge through small piping is subject to trouble from clogging. In a large plant now under design it is proposed to use an exchanger built of 6-inch sludge pipe around which will be an 8-inch hot water jacket (see Fig. 2). This will be installed in the pipe tunnel between the source of sludge and the sludge tanks. The pipe is large enough to prevent clogging, the system is enclosed and will not produce odors, can be cleaned with steam or hot water, is accessible, and its use helps to keep the sludge tank clear. Equipment for direct application of steam is to be provided for use when insufficient heat can be obtained from waste heat from engines.

Conclusion

In this discussion of problems in handling sludge, specific statements or detailed recommendations on how problems should be solved have purposely been avoided. The wide range in characteristics of sludge and the differences in conditions and requirements at various treatment plants make it difficult to draw general conclusions from experience. An effort has been made to call attention to a few experiences and to comment upon some difficulties encountered and the remedies which have been used to overcome them. Additional descriptions of experiences, particularly relating to procedures which have coped successfully with sludge handling problems, will be helpful.

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AIR RELEASE FOR SEWAGE PUMPS

Solves Pumping Problem

By W. W. GLOVER

Chief Engr., Simplex Valve & Meter Co., Philadelphia, Pa.

THE continuous and satisfactory service of sewage pumping units is very frequently affected by alternating pressure and vacuum conditions, due to automatic pump starting and stopping. This condition poses the problem of maintaining at all times the priming of the pumps. A particular case in point is that of the low-lift sewage pumping station at Pottstown, Pa.

At the Pottstown plant there are two pumps that deliver sewage from the low-level system, comprising approximately one-half of the sewerage system of the town, to the junction manhole approximately 1,000 ft. distant. From that point the total flow enters the treatment plant. The pumps deliver against a head of approximately 45 ft., taking suction from a common wet well in which float switches effect automatic operation. At the present time one pump has capacity sufficient for normal requirements so that the other is idle. One unit is operated for a 12-hour period during the day, while the other unit is operated the remaining 12 hours handling night flows for one week. The cycle is then alternated for the next week so that the two pump units have, in the long run, the same amount of service and the same types of flow to handle. When in service the pumping cycle provides an operating time of 12 to 15 minutes and an idle time of 6 to 8 minutes, except during the low flow hours of 12 midnight to 5 a.m., when the idle time is approximately 45 minutes.

The center line of the pump casing is at elevation 15.

The float switches are so set as to start the pump when the suction well level is at elevation 17 and stop it when the level has dropped to elevation 13. Once each day the wet well is drawn down to elevation 10.5 in order to clear it of the accumulating heavier solids. For general diagrammatic view see Fig. 1.

The Problem and Attempted Correction

Under the circumstances a vacuum is present in the pump casing a considerable portion of the time, tending to release air from the sewage or draw it in at glands and joints. Air or gas is also liberated in some amount during normal operation. It is, therefore, necessary to displace accumulating air from the pump during operation and to prevent the entrance of air during periods of shut-down and low wet well level. Such prevention was attempted by placing an automatic air release valve on each pump casing near its outlet, just ahead of the check valve (Fig. 1). Sewage with its varied makeup always presents problems to mechanical equipment, but this case was further complicated by the fact that usually there is a certain amount of grease and oil present and frequently there are large quantities of it.

The original valves were not able to cope with this condition and the result was a need for dismantling and cleaning them two or three times a day. It was also necessary to have a manual release valve for those times when the pumps had to go into service at once without permitting out-of-service time for cleaning. This was a serious problem, especially since the pumps are three floors below the operating level and the station is 1,200 ft. from the main part of the treatment plant. It took much valuable time of the operating personnel to do all of this.

Problem Solved

The problem was presented to the writer's company and it was recommended that a Model B Simplex sewage air release valve be installed in these locations. The valves were located in the same place (Figs. 1 and 2) as the original valves and serve under the same operating conditions. These air release valves have now been in service for several months and are giving entire satisfaction. Not once in this period have the pumps lost their prime. On one occasion during this time a float switch failed to function and the wet well was drawn down to the bottom of the suction pipe and held there approximately for three-quarters of an hour. This brought a large quantity of floating grease through the pump and unusual quantities of air. This all was handled, however, without affecting the pumping efficiency thereafter.

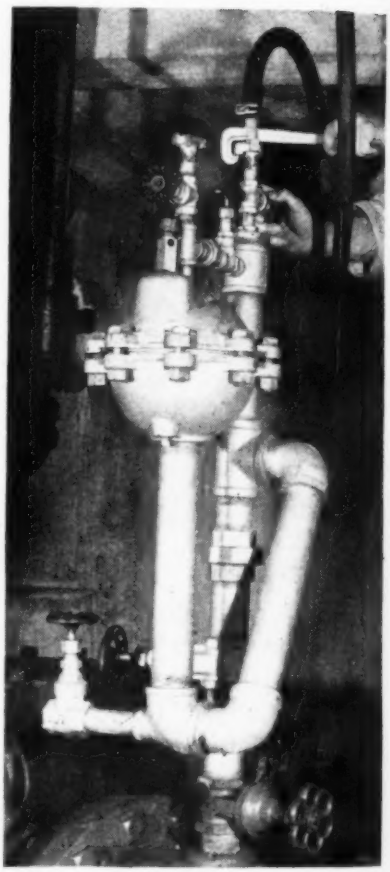


Fig. 2 — Sewage Air Release Valve Being Back Flushed

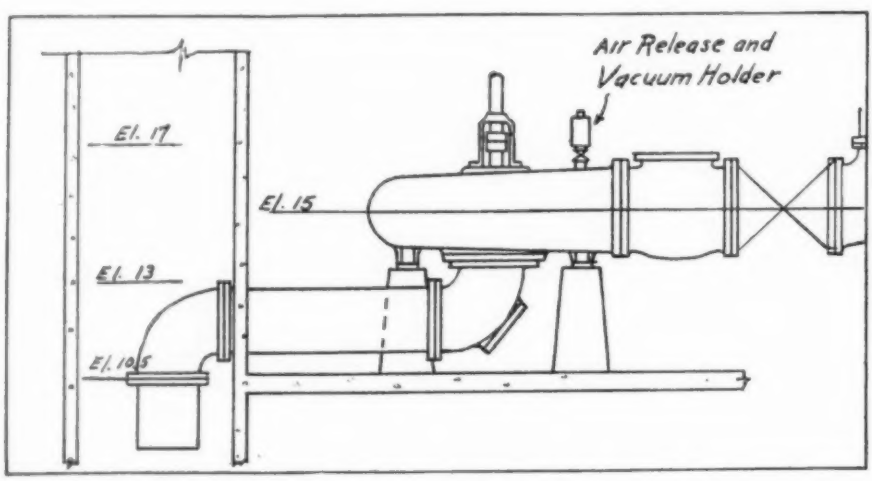
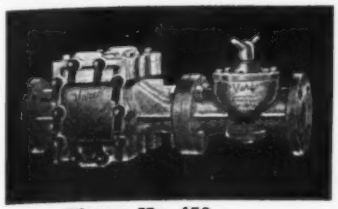


Fig. 1—Diagrammatic Sketch of Pump, Wet Well Elevations and Location of Air Release and Vacuum Holding Valve

Maintenance consists of flushing with clear water using the disconnector fitting shown in Fig. 2. Any sediment that carries over has an opportunity to settle and be flushed out at a lower blow-off valve. Air is automatically vented whenever the pump casing is under pressure, and a vacuum seal at the valve outlet prevents air from being drawn back into the valve or casing during shut-down periods.

With this arrangement in service, maintenance has been reduced to one operation per week resulting in a time saving of approximately 10 hours each week. This is an important consideration at any time, but especially now, with the great shortage in manpower. It is expected that an analysis of future pump operation will show very definite power savings because of the elimination of those occasions when the pump goes into operation with the casing largely filled with air, resulting in a useless churning of the pump without producing any sewage flow.

"Varec" THE GUARDIAN OF SEWAGE GAS CONTROL



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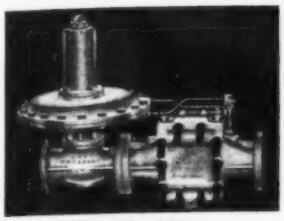

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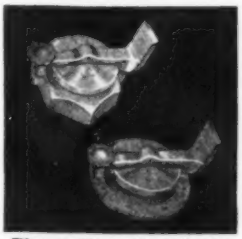

Figure Nos. 48 and 42A

Figure No. 220A

"Varec" Approved Pressure Relief and Flame Trap Assembly

Unit consists of Diaphragm-operated Regulator. Flame Trap, and Thermal Shut-off Valve. Maintains a predetermined back pressure, passing surplus gas to Burner. Stops flame propagation. Patented telescopic flame trap element simplifies inspection and maintenance. Listed by the Underwriters' Laboratories for vent lines in oil tanks.

Sizes	2"	2½"	3"	4"	6"
Approx. wt.	125	140	160	200	260

"Varec" Approved Flame Trap Assembly

Flame Trap and Thermal Shut-off Valve Unit self-contained, simple, foolproof. Installed in all gas lines supplying gas utilization equipment. No expensive fitting-up necessary. Arrests flame propagation. Easy inspection and maintenance. Aluminum and stainless steel. Non-corrosive. Flame Trap Element listed by Underwriters' Laboratories for vent lines in oil tanks.

Sizes	2"	2½"	3"	4"	6"	8"
Approx. wt.	90	100	115	150	200	300

"Varec" Approved Sensitive Pressure (Reducing) Regulator Single Port

Maintains upstream or downstream pressure to within 0.5" of water of pre-determined pressure. Positive Shutoff. Aluminum body. Throttling type inner valve. 18-8 stainless steel trim. Sizes 2" to 6".

Sizes	2"	2½"	3"	4"
Approx. wt.	110	115	140	175

"Varic" Approved Super Sensitive Pressure (Reducing) Regulator Double Port

Maintains upstream or downstream pressure to within 0.2" of water of pre-determined pressure. Standard working parts 18-8 stainless steel with synthetic rubber diaphragm. Non-corrosive. Sizes ½" to 10".

Sizes	2"	2½"	3"	4"	6"
Approx. wt.	190	200	233	260	350

"Varec" Approved Sampling Hatch Covers

For use on digester domes. Available in several combinations of materials. Non-corrosive, gas-tight, self-closing, spark-proof. Figure 42A Flanged and Figure 48 Screwed.

Sizes	4"	6"	8"	10"
Approx. wt.	12	17	23	27

"Varec" Approved Manhole Covers

Installed on digester and gas holder dome, affords quick and easy access. Has square, graphite-impregnated, fireproof gasket. Non-sparking. Gas-tight.

Sizes	18"	20"
Approx. wt.	100	110

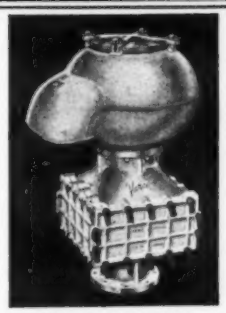

Figure No. 58C

"Varec" Approved Pressure Relief and Vacuum Breaker Valve With Flame Arrester

Installed on digesters and gas holder domes, it affords emergency Pressure and Vacuum Relief and prevents flame entrance from atmospheric disturbances. Equipped with extensible Flame Arresting element for easy inspection and maintenance. Pure aluminum. Non-corrosive. Listed by the Underwriters' Laboratories for Vent Lines in oil tanks. Size 2" to 10".

Sizes	2"	3"	4"	6"
Approx. wt.	70	105	160	225

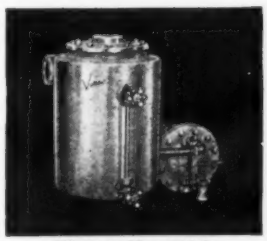

Figure No. 211A

"Varec" Approved Sediment Trap and Condensate Drip Trap Assembly

Cast iron construction. 18-8 stainless steel working parts. Hand operated or automatic. Sizes 2"-2½"-3"-4".

Figure No. 232D

"Varec" Approved Back-Pressure Check Valve

Installed in relatively low pressure gas lines. Prevents back flow through meters. Non-shattering, non-pulsating, non-corrosive, non-sparking.

Sizes	2"	3"	4"	6"
Approx. wt.	20	35	70	125

Figure No. 51A

"Varec" Approved Flame Check

Acid resisting Union type fitting. Positive flame stop for small lines. Has five 40-mesh fire screens. Easy inspection and maintenance.

Sizes	½"	¾"	1"
Approx. wt.	10	10	10

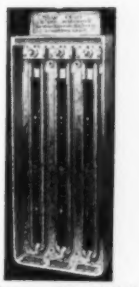

Figure No. 216A

Figure No. 236

"Varec" Approved Waste Gas Burners

Installed wherever disposal of surplus gas is a problem. Unit has adjustable air intake in the venturi tube, pilot valve adjustable from outside. Long, heavy cast-iron draft stack insures proper draft and complete combustion.

Sizes	2"	3"	4"
Approx. wt.	450	480	500

"Varec" Approved Manometers

Single or Triple tube. Open or push-button control types. Accurate. Aluminum housing. Bronze fittings. Pyrex glass. Automatic—Fireproof—Safe. Sizes 6" to 36".

Sizes	9"	12"	15"
Approx. wt.	16	18	20

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Cable: VAREC COMPTON (all codes)

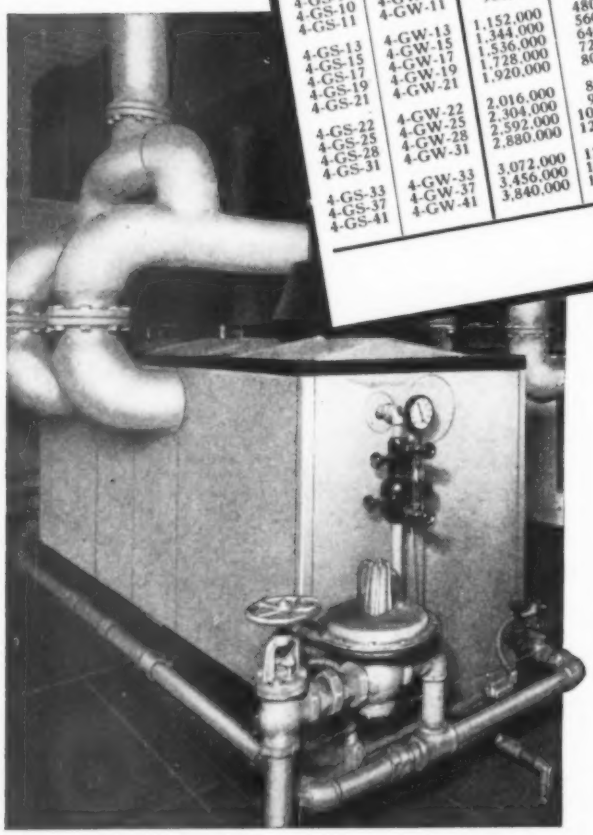
For heating sludge digesters

AMERICAN GAS BOILERS

AMERICAN GAS BOILER CAPACITIES WITH SEWAGE DISPOSAL GAS

BOILER NO.*		OUTPUT PER HOUR			INPUT PER HOUR		BOILER SPACE		
								Floor Space	
Steam	Water	BTU per Hour	Sq. Ft. Steam A.G.A. Rating	Sq. Ft. Water A.G.A. Rating	BTU per Hour	Cu. Ft 700 BTU Sludge Gas	Over-all Hgt. xx	Width Front	Depth
.....	1-GA-4E	31,500	.....	210	39,350	56	40½"	16"	19¼"
.....	1-GA-5E	42,000	.....	280	52,500	75	40½	16	22¼
.....	1-GA-6E	52,500	.....	350	65,600	94	40½	16	25¼
.....	2-GA-4E	63,000	.....	420	78,750	113	50½	16	19¼
.....	2-GA-5E	84,000	.....	560	105,000	150	50½	16	22¼
.....	2-GA-6E	105,000	.....	700	131,200	188	50½	16	25¼
.....	2-GA-7E	126,000	.....	840	157,500	225	50½	16	28¼
.....	4-GA-8E	147,000	.....	980	183,750	263	54	16	37¼
.....	4-GA-9E	168,000	.....	1120	210,000	300	54½	16	40¼
.....	4-GA-11E	210,000	.....	1400	262,500	375	54½	16	46¼
.....	4-GA-13E	252,000	.....	1680	315,000	450	56	16	52¼
.....	.....	61,200	255	.....	76,600	109	52¼	16	33¾
.....	.....	81,600	340	.....	102,000	146	53	16	37¾
.....	.....	102,000	425	.....	127,500	182	52½	16	41¾
0-GS-4E	.....	146,000	610	980	182,500	261	54½	35	25
0-GS-5E	.....	186,000	775	1240	232,500	332	56¾	39	25
0-GS-6E	.....	225,000	940	1500	281,500	402	56½	43	25
1-GS-4	1-GW-4	265,000	1105	1770	331,000	473	57	47	25
1-GS-5	1-GW-5	305,000	1270	2030	381,000	545	59¼	51	25
1-GS-6	1-GW-6	344,000	1435	2300	431,000	616	58½	55	25
1-GS-7	1-GW-7	384,000	1600	2560	480,000	686	59¼	59	25
1-GS-8	1-GW-8	424,000	1765	2820	530,000	757	59	63	25
1-GS-9	1-GW-9	480,000	2000	3200	600,000	857	74	53	41½
1-GS-10	1-GW-10	576,000	2400	3840	722,000	1030	75	58	41½
1-GS-11	1-GW-11	672,000	2800	4480	840,000	1200	77	62	41½
4-GS-6	4-GW-6	768,000	3200	5120	960,000	1372	77¾	66	41½
4-GS-7	4-GW-7	864,000	3600	5760	1,079,000	1543	79½	70	41½
4-GS-8	4-GW-8	960,000	4000	6400	1,200,000	1715	80	74	41½
4-GS-9	4-GW-9	1,152,000	4800	7680	1,440,000	2060	75	85	41½
4-GS-10	4-GW-10	1,344,000	5600	8960	1,680,000	2400	77	93	41½
4-GS-11	4-GW-11	1,536,000	6400	10,240	1,920,000	2740	77¾	103	41½
4-GS-13	4-GW-13	1,728,000	7200	11,520	2,162,000	3090	79½	107	41½
4-GS-15	4-GW-15	1,920,000	8000	12,800	2,400,000	3425	80	110	41½
4-GS-17	4-GW-17	2,016,000	8400	13,440	2,520,000	3600	77	136	41½
4-GS-19	4-GW-19	2,304,000	9600	15,360	2,880,000	4120	77¾	148	41½
4-GS-21	4-GW-21	2,592,000	10,800	17,280	3,240,000	4625	79½	160	41½
4-GS-22	4-GW-22	2,880,000	12,000	19,200	3,600,000	5145	80	172	41½
4-GS-25	4-GW-25	3,072,000	12,800	20,480	3,840,000	5480	77¾	186	41½
4-GS-28	4-GW-28	3,456,000	14,400	23,040	4,320,000	6165	79½	202	41½
4-GS-31	4-GW-31	3,840,000	16,000	25,600	4,800,000	6850	80	218	41½
4-GS-33	4-GW-33								
4-GS-37	4-GW-37								
4-GS-41	4-GW-41								

xx To top of Draft Hood



American Gas Boilers for heating sludge digesters are engineered for dependable operating and are especially adapted for burning sewage gas. They are completely automatic—one valve controlling water temperature to any predetermined degree. Pintype sections afford the most effective type of cast iron heat-absorbing surface. Controls are available to accommodate any desired system of gas, temperature, or pressure regulation. A wide range of sizes meets all requirements. For complete details of these performance-proved gas boilers, as well as information about American Gas Fired Automatic Storage Water Heaters, write American Radiator & Standard Sanitary Corporation, P. O. Box 1226, Pittsburgh 30, Pa.

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SEWAGE SLUDGE DIGESTION TANK HEATING

Some Theoretical and Practical Considerations in Design and Operation

By NORMAN C. WITTWER

Consulting Sanitary Engineer, Trenton, N. J.

THE artificial maintenance of optimum temperatures in sludge digestion tanks has become standard practice in sewage treatment plant design and operation in the northern portion of the United States, and in other localities subject to adverse climatic conditions. Experience has shown that the lowering of the temperature of digesting sludge in unheated or improperly heated digestion tanks during the winter months seriously reduces biological activity, thus contributing greatly to the operating problems inherent in sewage sludge digestion.

Most engineers and sewage treatment plant operators have reached the conclusion that digestion tank temperatures of from 80° F. to 90° F. are most satisfactory, both as to results obtained, and as to economy of heating and heat exchange. Some plants keep the temperature of digesting sludge well above 90° F., but this practice is the exception rather than the rule. The writer's experience with temperatures at or about 85° F. has, on the whole, been very satisfactory, and there seems to be very little justification for maintaining temperatures above this level. On the other hand, as temperatures drop below 80° F., the digestion of sludge is gradually retarded. When the temperature drops below 70° F., active digestion ceases almost completely, and the sludge lies dormant until the proper temperature environment is reestablished. The absence of proper temperature environment in the digestion compartments of Imhoff tanks explains in part some of the difficulties which have always been encountered with these units.

Proper Design Voids Most Operating Problems

The problem of providing heat for sludge digestion purposes is largely one of original design, although it becomes necessary at times to install heating facilities at existing plants. Usually, heat exchange equipment installed after a plant has been placed into operation is more or less a make-shift arrangement, and is not nearly so effective or economical as would be a heating system properly designed and installed at the time the plant was constructed. The responsibility for successful sludge digestion rests primarily with the designing engineer. If he realizes the importance of properly designed sludge heating, future operating troubles due to ineffective sludge digestion will be reduced to a minimum.

The technical solution of sludge digestion tank heating problems involves the same fundamental technique used in the solution of other heating and heat transmission problems. However, there has been some tendency on the part of sanitary engineers to ignore such fundamentals, and to use instead certain empirical formulae or values for the determination of the over-all heating requirements of sludge digestion tanks. These formulae or values have not been adequately substantiated by scientific study and experimentation for all of the varying types of construction and conditions met with in ordinary practice, and the indiscriminate use of such methods has naturally resulted in some plant installations which have been deficient in heating capacity or heat exchange, or which have been over-designed.

However, the careful designer will treat each heating application as an individual problem, involving a consideration of climate, digestion tank construction and exposure, all heat losses through the structure, fresh sludge quantities, and proper type of heat exchange equipment. When broken down into its component parts, the solution of such a problem becomes quite simple, involving only a rudimentary knowledge of the fundamentals of heat transmission.

This discussion will present the subject matter in the same order as would ordinarily be used in the solution of a practical problem in sludge digestion tank heating. Fundamental theory and illustrative problems will be given concurrently for each of the major considerations involved. These are:

1. Heat losses from a sludge digestion tank, involving the determination of heat losses from a heated liquid sludge mass to air or ground through the exposed structural components of the tank.
2. Heat required to raise the temperature of the additions of fresh sludge to the temperature of digesting sludge in a sludge digestion tank.
3. A consideration of methods for applying heat to the contents of a sludge digestion tank
4. Theory of heat transmission as applied to heat exchangers, and design applications.
5. Selection of heating equipment.

HEAT LOSSES FROM A SLUDGE DIGESTION TANK

Methods of Heat Transfer

Heat transmission (or heat losses) between the mediums on two sides of a wall, roof, or floor takes place by three methods, namely, radiation, convection, and conduction. These methods of heat transfer may be defined as follows:

Radiation is the transmission of heat through space by wave motion in a manner similar to the travel of light waves.

Convection of heat occurs in fluids (liquids or gases), and is the transfer of heat accomplished by the motion of the fluid from a locality where it receives heat to a locality where it gives up heat. The motion of the fluid is usually the result of differences in density caused by temperature differences. This is called free convection. If the motion is produced by some mechanical means, such as a stirrer, or a circulating pump, it is called forced convection.

Conduction is the transfer of heat from one part of a continuous material to another part of the same material, or from one material to another in physical contact with it, without appreciable movement of the particles of the material.

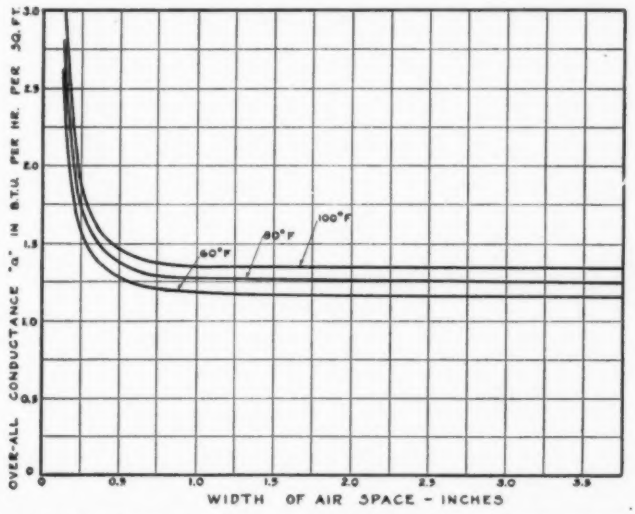

Fig. 1. Air Space Conductances at Several Mean Temperatures

In most cases the actual transfer of heat is accomplished by more than one of the above defined methods. It is therefore preferable to use other terms, such as "transfer," or "transmission," or "conductance," to describe an over-all heat transfer process, reserving the use of the terms "radiation," "convection," and "conduction" for that fraction of the total heat transfer accomplished by the mechanism designated.

For example, consider a wall constructed of one thickness of concrete, an air space, and a second thickness of brick masonry with air on both sides of the composite wall, and with the concrete being adjacent to the warmer air. Heat will be received by the concrete surface from the inside air by radiation, convection and conduction. It will be carried through the homogeneous concrete section by conduction, and carried across to the opposite brick wall surface through the air space by radiation, convection and conduction. From here it will be carried by conduction through the homogeneous brick section to the outer surface of the wall, and will leave this surface by radiation, conduction and convection.

Any digestion tank wall, roof, floating cover, gas dome or bottom floor may be divided into internal and external surfaces, homogeneous materials, and interior air or gas spaces, as the

case may be. Practical heat transmission coefficients have been determined experimentally which will give the total heat transferred through any of these component parts of a structure, and if the method of applying these individual coefficients is thoroughly understood, it is a simple matter to calculate the over-all heat transmission or heat losses through any combination of materials.

Notation Used in Heat Loss Formulae

- k = Thermal conductivity of a homogeneous material 1-inch thick, in B.t.u./hr./sq. ft./1° F. difference in temperature between the two surfaces of the material.
- f = Film or surface conductance; the amount of heat expressed in B.t.u./hr./sq. ft./1° F. temperature difference transmitted by convection or conduction from a fluid to a surface with which it is in contact, or vice versa, from a surface to the fluid surrounding it.
  - f_i = coefficient of heat transfer *to* the inner or warm surface.
  - f_o = coefficient of heat transfer *from* the outer or cold surface.
- a = Thermal conductance of an air or gas space in B.t.u./hr./sq. ft./1° F. temperature difference.
- U = Thermal conductance or over-all coefficient of heat transmission; the amount of heat transmitted in B.t.u./hr./sq. ft./1° F. difference in temperature between the warm and cold sides of a wall, roof or floor.
- H = Total heat loss in B.t.u./hr. transmitted through any material, or any combination of materials, such as a wall, roof, cover, or floor of a sludge digestion tank for the total area involved.
- A = Area of a surface in square feet as actually measured from the plans or from the structure. Use the net inside or heated dimensions.
- T_i = Temperature in °F. of the medium adjacent to the inside or warmer surface; the inside temperature.
- T_o = Temperature in °F. of the medium adjacent to the outside or colder surface; the outside temperature.
- b = Thickness in inches of homogeneous materials.

Heat Flow by Conduction

The flow of heat through a homogeneous material by conduction is proportional to

(a) the thermal conductivity of the material.
(b) the difference in temperature between the two surfaces or planes considered.
(c) the area through which the heat flows.
and (d) inversely proportional to the distance between the two surfaces or planes considered.

Thus, the total heat transferred between two surfaces of a homogeneous material by conduction can be expressed by the following formula:

$$H = \frac{A\,k\,(T_i - T_o)}{b} \qquad (1)$$

The coefficients of thermal conductivity given below in Table 1 for homogeneous materials are average values, and have been shown by numerous experiments to be accurate for the typical materials described. Since the conductivity of a material has been found to depend upon its composition, texture and density, these coefficients should not be used for other variations of these materials. Other coefficients can be found in standard reference works.

TABLE 1.

THERMAL CONDUCTIVITY COEFFICIENTS "K"

For Homogeneous Materials (1" Thick) in B.t.u./hr./sq. ft./°F.

Material	k
Cement mortar	12.00
Brickwork, low density	5.00
Brickwork, high density	9.20
Concrete, typical stone structural	12.00
Stone masonry	12.50
Wood, average value for ordinary woods	1.00
Asphalt or tar composition roofing	6.50*
Wrought iron	370.
Steel (1% C)	314.
Pure copper	2640.
Water	4.10

*For thickness used on typical roof construction, not per 1-inch of thickness.

Film Conductance

When gaseous fluids such as air or sewage gas are in contact with solids, heat will flow from the warmer to the colder material. The method of heat transfer is principally a combination of convection and conduction. However, the condition of the air or gas has a very important influence on the rate and mode of heat transfer. It is certain that in all cases a stationary film of air or gas is present on the surface of the solid material, separating the solid from the main body of air or gas. The thickness of this film depends upon its temperature, and upon the velocity of the air or gas parallel to the solid surface.

Therefore, heat transfer through an air or gas film has been found to proceed at a higher rate where movement or velocity exists than is the case for still air. This is illustrated by the film coefficients in common use by engineers for general heating practice. These coefficients are not for convection alone, or for conduction alone, but are experimental, combination over-all film coefficients for heat transfer from a wall surface to or from the main body of adjacent air. They are as follows:

- f_i = 1.65 B.t.u./hr./sq. ft. for inside air without appreciable motion or velocity.
- f_o = 6.00 B.t.u./hr./sq. ft. for outside air with a velocity of 15 miles per hour or higher.

The above coefficients are sufficiently accurate for air or gas film conductance as it exists on the inside and exterior surfaces of sludge digestion tank structures, and will be used throughout this discussion. They will be applied to sewage gas (approx. 30 per cent CO_2 and 70 per cent CH_4) as well as to atmospheric air.

Where liquids are in contact with a structure through which heat transmission takes place, film resistance is also present between the surface of the structure and the main body of the liquid. However, as will be explained in some detail later on in connection with the discussion on heat exchangers, the film coefficient is influenced to a large degree by the motion of the liquid. Where temperature differences between liquid and wall surface are small, natural or free convection will be slight, and very little motion of the liquid adjacent to the walls of a digestion tank will be present. Therefore for practical purposes, both the liquid contents of a tank, and the dry or wet earth surrounding the tank, are assumed to be always in direct contact with the structure, without an intermediate film. However, where either dry or wet earth is in contact with the exposed, cold side of the structure, a certain amount of resistance to the transmission of heat is present. While not strictly film resistance, such resistance is of some significance. Therefore, the coefficients for heat transfer from wall surfaces into earth are included in Table 2.

TABLE 2.

FILM COEFFICIENTS "F"

(B.t.u./hr./sq. ft. °F.)

	k
Concrete to or from still air	1.65
Concrete to outside moving air (15 mi. per hr.)	6.00
Brickwork to or from still air	1.65
Brickwork to outside moving air (15 mi. per hr.)	6.00
Steel to or from still air	1.00
Steel to outside moving air (15 mi. per hr.)	3.00
Composition roofing to outside moving air	10.50
Concrete to dry earth	1.00
Concrete to wet earth or to water	2.00

Air and Gas Spaces

Heat is carried across an air or gas space by a combination of radiation, convection or conduction. A large part of the heat transferred across an air space bounded by ordinary building materials such as wood, plaster, concrete, etc., is by radiation. The effect of heat radiation can be greatly reduced by facing the air spaces with metallic surfaces such as aluminum foil, coated sheet steel, or other low-emissivity, infra-red reflective metal surfaces, thus causing a major portion of the heat to be transmitted by convection. In sludge digestion tank construction, only ordinary building materials are customarily used, and we are therefore concerned only with air spaces bounded by such materials, without reflective surfaces.

It has been found as the result of experiments conducted at the University of Minnesota under a cooperative research agreement with the American Society of Heating and Ventilating Engineers that the over-all thermal conductances across air spaces between ordinary building materials are fairly uniform at a given mean temperature for widths of air space in excess of ¾-inch. This is illustrated by Figure 1, in which curves for several mean temperatures show the relationship between widths of vertical air spaces and over-all surface to surface conductances.

From these curves, an average conductance value of 1.25 B.t.u./hr./sq. ft./1° F. temperature difference is established for vertical air spaces ¾-inch or more in width, and for the temperatures ordinarily encountered in sewage sludge digestion practice. When air spaces are not faced with reflective insulating materials, the difference between upward heat flow through single horizontal or sloping air spaces and through single vertical air spaces is comparatively small for the same temperature difference. Therefore, the same conductance value of 1.25 will be used for all air and gas spaces found in sludge digestion tank construction, whether they be vertical or horizontal.

Resistance Concept

It is obvious from the above discussion of heat transmission that the higher the thermal conductivity or over-all conductance of a material or combination of materials, the lower is its resistance to the flow of heat. Thus, the resistance to the flow of heat is inversely proportional to the thermal conductivity or conductance. If R is used to designate over-all resistance, and r_a

r_s, etc., are used to designate unit resistances, then the over-all resistance to the flow of heat through a structure will be

$$R = r_1 + r_2 + \ldots\ldots + r_n \quad \ldots\ldots (2)$$

If Equation (2) is applied to a wall built up of a single homogeneous material b inches thick, then

$$\frac{1}{U} = \frac{1}{f_i} + \frac{b}{k} + \frac{1}{f_o} \quad \ldots\ldots (3)$$

where $\frac{1}{U} = R =$ over-all resistance.

$\frac{1}{f_i} = r_i =$ inside film resistance.

$\frac{1}{f_o} = r_n =$ outside film resistance.

$\frac{b}{k} = r_2 =$ internal resistance to heat transfer by conduction.

For a compound wall built up of two homogeneous materials having thicknesses b_1-inches and b_2-inches, and conductivity coefficients k_1 and k_2 respectively, separated to form an air space of conductance a, and with both sides of the wall exposed to air, the over-all resistance will be

$$\frac{1}{U} = \frac{1}{f_i} + \frac{b_1}{k_1} + \frac{1}{a} + \frac{b_2}{k_2} + \frac{1}{f_o} \quad \ldots\ldots (4)$$

where $\frac{1}{a}$ = air or gas space resistance. The over-all coefficient of heat transmission U then becomes

$$U = \frac{1}{\frac{1}{f_i} + \frac{b_1}{k_1} + \frac{1}{a} + \frac{b_2}{k_2} + \frac{1}{f_o}} \quad \ldots\ldots (5)$$

If the inside air is replaced with water then $\frac{1}{f_i}$ in Equations (4) and (5) becomes zero.

Heat Loss Formulae

Referring back to Equation (1), the same relation exists for a compound wall as is the case for a single homogeneous material. Therefore, the total heat loss between two surfaces of any structure can be expressed by

$$H = A\ U\ (T_i - T_o) \quad \ldots\ldots (6)$$

If the proper value for U is substituted in Equation (6), as for example the U in Equation (5), then the total heat loss H becomes,

$$H = \frac{A\ (T_i - T_o)}{\frac{1}{f_i} + \frac{b_1}{k_1} + \frac{1}{a} + \frac{b_2}{k_2} + \frac{1}{f_o}} \quad \ldots\ldots (7)$$

Equation (7) is the fundamental formula for the total heat loss through any material or combination of materials. In applying this formula, it should be remembered that each material in a compound wall, roof or floor will have a different coefficient k, and that there will be as many values of $\frac{b}{K}$ as there are different materials.

Computation of Sludge Digestion Tank Heat Losses

The following problem will illustrate how the fundamental formulae and heat flow coefficients may be applied to the determination of heat losses from sludge digestion tanks. A common type of construction will be shown, and the method used in solving this problem can be extended to almost any type of digestion tank design. Since all such calculations are at the best only approximate, the slide rule will be used throughout.

Illustrative Problem 1.

Calculate the total heat loss, in B.t.u. per hour, for the sludge digestion tank shown in Figure 2. The inside sludge temperature will be 85° F.; the minimum sustained winter air temperature is estimated at 15° F.; the minimum sustained dry earth temperature at 40° F.; and the minimum sustained wet earth temperature at 50° F. For the purpose of this computation, heat losses from the scum-covered annular ring of water surface between the floating cover and the tank walls will be ignored, as this loss will be very small, and will be more than compensated for by assuming the diameter of the floating cover to be equal to the inside diameter of the tank.

The total heat loss will be the sum of the individual losses through the tank floor, the several vertical sections of the wall,

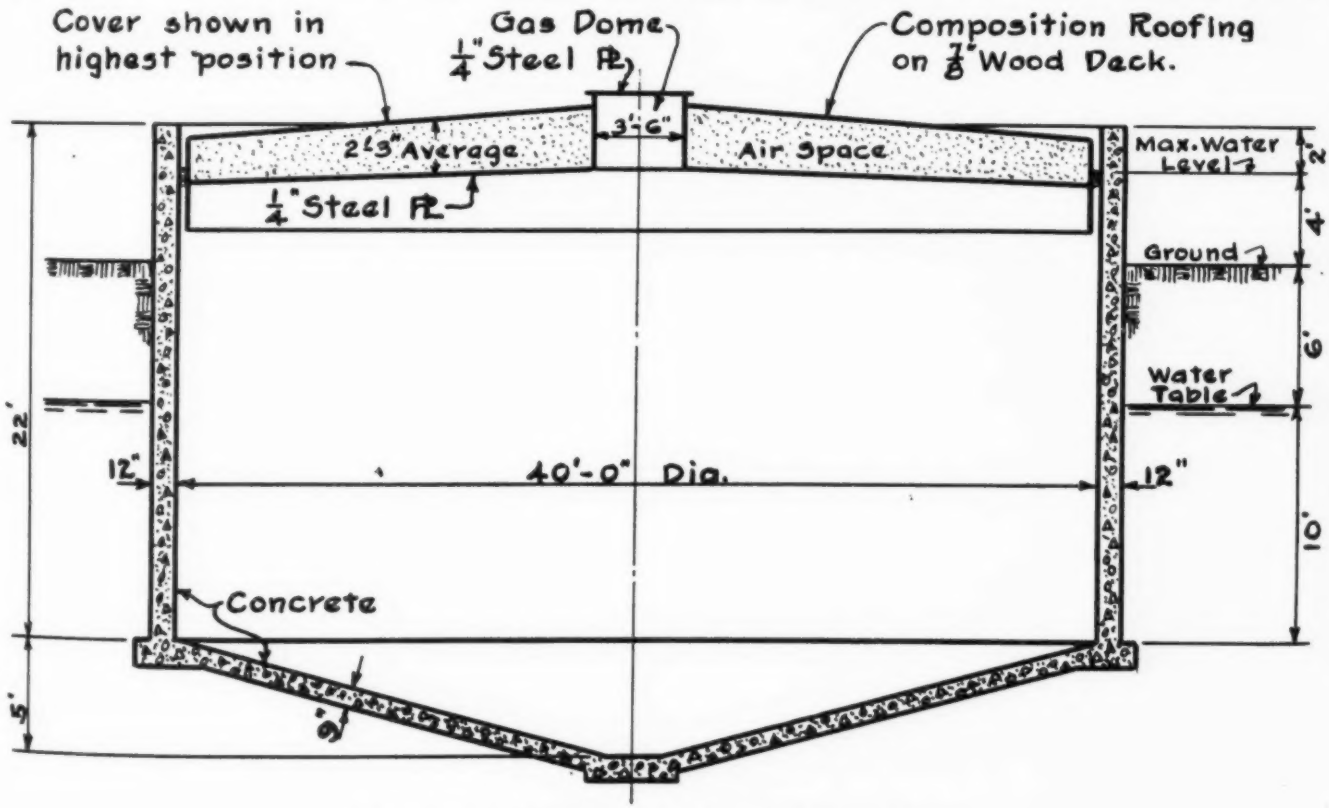

Fig. 2. Typical Sludge Digestion Tank with Floating Cover

and the floating cover. As stated above, where inside water or sludge is in direct contact with the structure, the resistance $\frac{1}{f_i}$ will be zero.

Solution

1. Heat loss H_1 through the tank floor. This involves only the heat transfer through the 9-inch thick concrete slab, and from the bottom of the slab into the wet earth.

$$H_1 = \frac{A\ (T_i - T_o)}{\frac{1}{f_i} + \frac{b}{k} + \frac{1}{f_o}}$$

$$= \frac{1290 \text{ sq. ft. } (85 - 50)}{0 + \frac{9}{12.00} + \frac{1}{2.00}}$$

$$= 36{,}200 \text{ B.t.u./hr.}$$

2. Heat loss H_2 through portion of wall below ground water table. This calculation is similar to that for H_1.

$$H_2 = \frac{1260 \text{ sq. ft. } (85 - 50)}{0 + \frac{12}{12.00} + \frac{1}{2.00}}$$

$$= 29{,}400 \text{ B.t.u./hr.}$$

3. Heat loss H_3 through portion of wall between ground water table and ground surface. This calculation is similar to that for H_1, except that heat is transferred into dry earth instead of into wet earth.

$$H_3 = \frac{755 \text{ sq. ft. } (85 - 40)}{0 + \frac{12}{12.00} + \frac{1}{1.00}}$$

$$= 17{,}000 \text{ B.t.u./hr.}$$

4. Heat loss H_4 through portion of wall exposed to outside air. This calculation involves the use of the common film coefficient from concrete to outside air. Otherwise, the method is the same as for H_1.

$$H_4 = \frac{502 \text{ sq. ft. } (85 - 15)}{0 + \frac{12}{12.00} + \frac{1}{6.00}}$$

$$= 30{,}000 \text{ B.t.u./hr.}$$

5. Heat loss H_5 through gas dome. This calculation involves the transfer of heat across the gas space, through the inside gas film, through the very high conductance of the steel plate and through the outside air film. It is evident that the resistance to the transfer of heat which is offered by the steel plate could be ignored without affecting the slide-rule result.

$$H_5 = \frac{A\ (T_i - T_o)}{\frac{1}{a} + \frac{1}{f_i} + \frac{b}{k} + \frac{1}{f_o}}$$

$$= \frac{10 \text{ sq. ft. } (85 - 15)}{\frac{1}{1.25} + \frac{1}{1.65} + \frac{0.25}{314} + \frac{1}{6.00}}$$

$$= 500 \text{ B.t.u./hr.}$$

6. Heat loss H_6 through floating cover, diameter taken as 40 feet. Four layers of material are involved, namely, steel, air, wood deck, and composition roofing.

$$H_6 = \frac{A\ (T_i - T_o)}{\frac{1}{f_i} + \frac{b_1}{k_1} + \frac{1}{a} + \frac{b_2}{k_2} + \frac{b_3}{k_3} + \frac{1}{f_o}}$$

$$= \frac{(1260 - 10) \text{ sq. ft. } \times (85 - 15)}{0 + \frac{0.25}{314} + \frac{1}{1.25} + \frac{0.875}{1.00} + \frac{1}{6.50} + \frac{1}{6.00}}$$

$$= 43{,}800 \text{ B.t.u./hr.}$$

7. The total heat loss becomes—

$$H_t = H_1 + H_2 + H_3 + H_4 + H_5 + H_6$$

$$= 156{,}900 \text{ B.t.u./hr.}$$

HEAT REQUIRED BY FRESH SLUDGE ADDITIONS

Method

The heat required to raise the temperature of the fresh sludge additions to the temperature of the mass of digesting sludge in the tank is obtained by a simple calculation. For practical purposes, one B.t.u. is the amount of heat required to raise the temperature of one pound of water one degree Fahrenheit. Sludge has approximately the same characteristics as has water. Therefore the

$$\text{Heat required} = \frac{W\ (T_s - T_i)}{24} = \text{B.t.u./hr.} \quad \ldots\ldots\ldots\ldots\ (8)$$

where

W = Weight in pounds per day of fresh sludge additions.
T_s = Temperature in °F. of fresh sludge, ordinarily the same as the sewage flowing through the plant.
T_i = Temperature in °F. of the digesting sludge in the digestion tank.

Quantity of Fresh Sludge Additions

A sewage treatment plant which has been in operation for some time should have available a back-log of operating records which will indicate the average daily volume of liquid sludge additions to the digestion system. Wherever possible, such records should be utilized in the calculation of digestion tank heating requirements. Where such records are not available, or in the case of a new plant design, the engineer must carefully estimate the probable quantities of fresh sludge to be expected, basing the estimate on his own experience, or on the experience of others.

The quantity of fresh sludge added to a sludge digestion tank is influenced by the character of the sewage, the type of sewage treatment process, the method of plant operation, and also to a large extent by the degree of care exercised by the plant operators in limiting fresh sludge pumpings to the thicker, more concentrated liquid. Obviously, the addition of large quantities of excess water to the sludge digestion tank may not only overtax the heating facilities, but may also seriously interfere with the normal functioning of the sludge digestion processes.

The average daily quantities of fresh sludge solids to be expected at different types of sewage treatment plants are shown in Table 3. These quantities are for domestic sewage, without an appreciable proportion of industrial wastes, and without storm water drainage. In using these average figures for design, some allowance should be made for higher-than-average rates which may be sustained over periods of time long enough to influence over-all heat requirements. The writer believes that an increase of 20 per cent above these values provides a safe allowance for all practical purposes, unless more definite information is available.

TABLE 3
AVERAGE DAILY QUANTITIES OF FRESH LIQUID SOLIDS ADDED TO SLUDGE DIGESTION TANKS

	Average Moisture	Sludge to Digester, cu. ft./day/capita		
	%	High	Aver.	Low
Primary Treatment Plants	95	.070	.050	.035
Chemical Precipitation Plants..	92.5	.125	.085	.070
Trickling Filter Plants*	94	.080	.060	.045
Activated Sludge Plants	98.5	.280	.150	.100

*Including Humus Tanks.

Illustrative Problem 2.

Calculate the heat required by the fresh sludge additions to the sludge digestion tank shown in Problem 1. This tank forms part of a primary sewage treatment plant, and is designed for 16,000 persons at 3 cu. ft. per capita. The minimum sustained fresh sludge temperature T_s is expected to be 45° F. From Problem 1, $T_i = 85°$ F.

Solution

The average daily quantity of fresh sludge additions is estimated from Table 3 at .050 cu. ft./day.

$$W = .050 \times 16{,}000 \times 62.5 \times 1.20\% = 60{,}000 \text{ lbs./day}$$

In Equation (8), heat required $= \frac{60{,}000\ (85 - 45)}{24}$

$$= 100{,}000 \text{ B.t.u./hr.}$$

Total Heat Required

It can be seen by the foregoing discussion that the total heat required for a sludge digestion tank is the sum of heat required to compensate for the heat losses from the liquid through the walls, roof and floor of the tank, and the heat required to raise the temperature of the incoming fresh sludge to the temperature of the digesting sludge. Thus, for the digestion tank of Problems 1 and 2, the total heat required will be

Heat loss through structure..	=	156,900 B.t.u./hr
Heat required for fresh sludge	=	100,000 B.t.u./hr.
Total heat required		256,900 B.t.u./hr.

METHODS FOR HEATING SLUDGE DIGESTION TANKS

Having determined the total heat requirements for a sludge digestion tank, the final element which enters into the thermal design of its heating system is the selection and calculation of an adequate heat exchange system. Several methods may be used for transferring heat into the liquid contents of a tank.

However, some methods, such as the use of steam or hot air in heating coils, are inapplicable to this type of problem since the resulting high temperature of the heat exchangers would be too conducive to scale formation. The methods worthy of our consideration are as follows:

1. Circulation of hot water through horizontal pipes placed around the inside periphery of a tank.
2. Circulation of hot water through vertical banks of heating pipes suspended in the liquid.
3. Direct introduction of hot water.
4. Preheating the fresh sludge before it is added to a tank.

Circulation of Hot Water Through Horizontal Pipes.—This is the most common method of transferring heat into a sludge digestion tank. The usual arrangement is to install the heating piping in the form of a spiral coil around the inside periphery of the tank as shown by Figure 3.

The spacing of the successive turns of the coil is usually about 12 inches. Hot water is admitted to the top of the coil and the colder return water is taken away from the lowest point.

In some instances, heating coils have been placed at or very near the floor of the tank. This practice is not too good because it tends to keep the more thoroughly digested sludge at a high temperature while the actively digesting sludge higher up in the tank receives only the benefit of somewhat lower temperatures. The most favorable location for the heating coil seems to be from a level about 6 ft. above the floor to a level not above the middle of the tank. This range usually covers the region of active sludge digestion.

Circulation of Hot Water Through Vertical Banks of Pipes.—Another common type of heat exchanger for digestion tanks with fixed roofs or covers is the vertical, multiple-bank type suspended from the roof. This type of heat exchanger is, or should be, removable from above. The vertical pipes are in series and are placed very close together, from 5 to 7 inches on centers. This type of heat exchanger is illustrated in Figure 4.

The hot water supply is introduced into the top of the first pipe, and is forced down and up through succeeding pipes until it is removed from the top of the last pipe. Such a unit should be located at or near the center of a small tank; while several units, distributed uniformly, should be used in larger tanks.

Direct Introduction of Hot Water.—The practice of introducing hot water directly into a digestion tank is of German origin, and up-to-date has not found much favor in this country. Several installations have been reported, but not much data as to their efficiency is available.

Keefer[1] found at Baltimore in comparative plant-scale tests that more heat was required to heat the sludge by the direct addition of hot water than was the case with well designed heat exchangers using circulated hot water. He claims, however, that the cost of constructing a pipe coil and return piping to the heating plant is saved; also since water at considerable pressure is generally available, it can be added directly to the tank without the use of pumps, whereas hot water circulating through the pipe coils must be pumped. But it must be remembered that the direct connection of a high pressure water supply, if potable, constitutes a cross-connection which is distinctly contrary to standard sanitary practice and health codes. Furthermore, the actual pumping of circulated water is a minor operation, involving very small pumps and low operating costs.

Downes[2], on the other hand, asserts that the practice of introducing hot water directly into the sludge contents of a digestion tank is detrimental as it tends to segregate grit and coarse particles such as seeds and inorganic materials from the sludge mass, thus accelerating accumulations of such materials in the bottom of the tank.

It is the writer's opinion that the objections to the direct addition of previously heated water more than outweigh the doubtful economy in first cost. Where outside water is used, the cost of purchase is in itself a continuing expense. In addition to the objections cited by Downes, the continuous introduction of hot water into the tank tends to thin out the sludge, thus reducing the amount of space available for digestion purposes. Naturally, there is a concurrent interference with sludge concentration, which is exactly the reverse of the above procedure.

Preheating the Fresh Sludge.—This method has been tried at a few plants. While not much operating data is available, it seems to the writer that the preheating of fresh undigested sludge would introduce an entirely new set of difficulties, not the least of which would be continuous sludge encrustation on the outside or inside of any heat exchanger used. This is due to the fact that the fresh sludge would necessarily have to be heated to a temperature considerably higher than the tank temperatures because it would be necessary to overcome all heat losses from the structure in addition to the mere raising of the fresh sludge to the temperature of the digesting sludge.

More experience with this method of heat exchange may ultimately demonstrate its fitness, both as to economy and operating effectiveness.

SLUDGE DIGESTION TANK HEAT EXCHANGERS

The theory involved in heat transfer from hot water circulated through spiral pipe coils around the inside of a digestion tank is quite similar to that involved in heat transfer from hot water circulated through banks of pipes suspended vertically into the liquid contents of a tank. Both types of heat exchangers involve a consideration of fluid motion, and of heat transfer by convection and conduction. As will be shown, the resistance in thin walled pipes and tubing to heat transfer by conduction is often much less than the resistance through the fluid films immediately in contact with the pipe walls. Film resistance is to a large extent influenced by convection, although the actual passage of heat through such a film is principally by conduction.

Consider the case of a fluid in contact with the wall of a pipe whose temperature is lower than that of the fluid, as shown by the left-hand portion of Figure 5. Although the main body of the fluid passing through the pipe may or may not be in turbulent motion, the thin film of fluid in contact with the pipe is relatively stagnant as the result of friction with the wall. The thickness of this stagnant film is not clearly defined and varies as the degree of turbulence of the passing fluid. Since the heat is transferred through this film largely by conduction rather than by convection, the entire process by which heat is transferred to the surface of the pipe from the main body of the fluid is somewhat complicated. Calculations are simplified, however, by the use of so-called *convection coefficients* which include the combined effect of both conduction and convection.

For digestion tank heating with pipe heat exchangers, it is

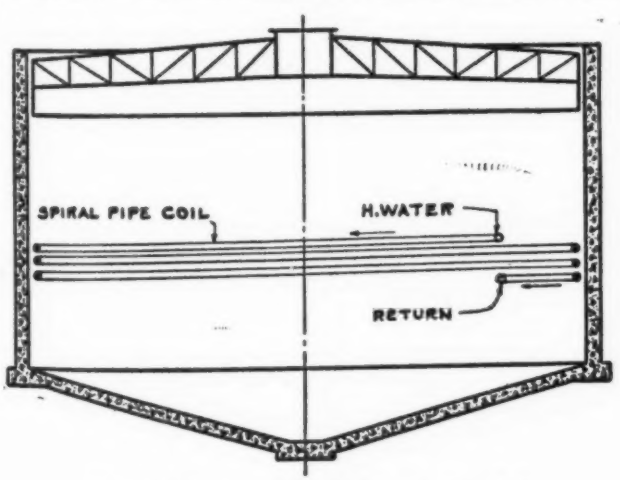

Fig. 3. Digestion Tank with Spiral Pipe Coil

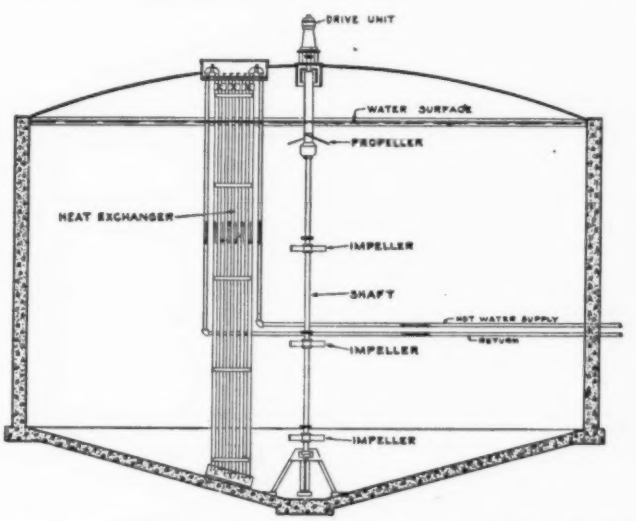

Fig. 4. Digestion Tank with Vertical Bank Type of Heat Exchanger

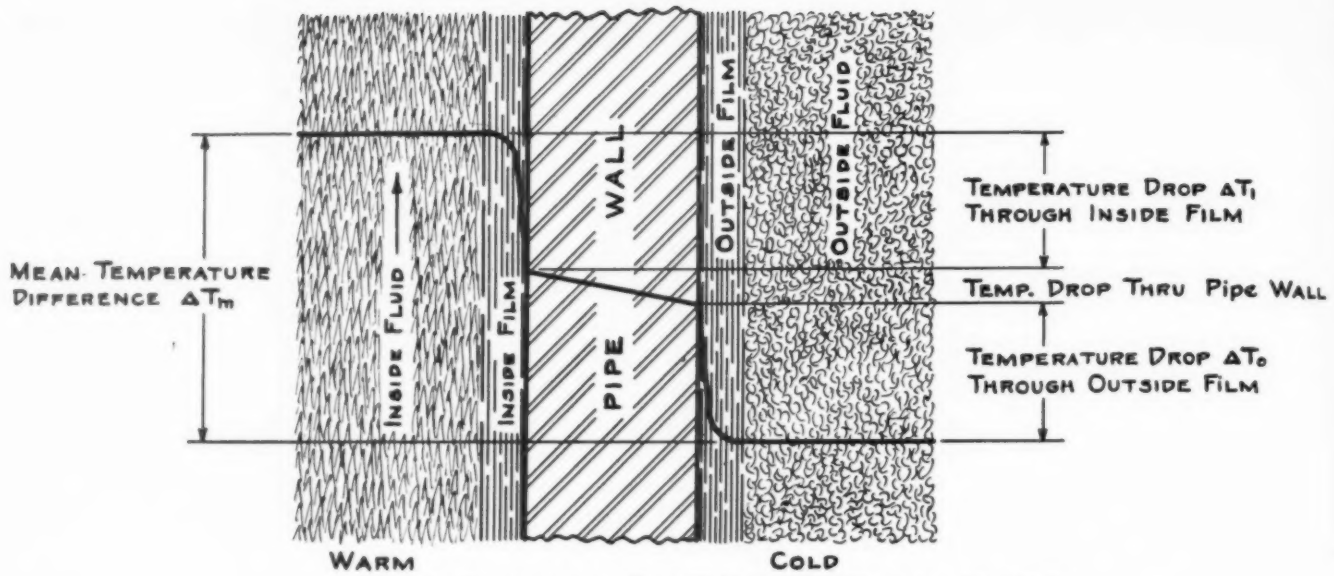

Fig. 5. Temperature gradient, Fluid to Fluid, Through a Pipe Wall

assumed that a state of equilibrium has been reached; i. e., the temperature gradient is assumed to remain constant and not change with time, and the rate of heat transfer through the wall of the pipe is assumed equal to the rate at which heat is absorbed by the liquid outside of the pipe. In addition, the effect of radiation is neglected, since the rate of heat transfer by radiation will be negligible compared to the rate of heat transfer by convection.

Film Coefficient.—From the above discussion it can be seen that there is no difference between this over-all film coefficient and the film coefficient f previously defined under "Notation used in heat loss formulae." It can be conveniently re-defined as the rate of heat transfer by convection between the inner or outer surface of a pipe wall and the main body of the adjacent fluid, in B.t.u./hr./sq. ft.°/F. temperature difference. Thus,

$$H_c = A\,f\,(\Delta T) \qquad (9)$$

where H_c = rate of heat transfer by convection, B.t.u./hr.
f = film coefficient, B.t.u./hr./sq. ft. °F.
A = area of wall surface in sq. ft.
ΔT = temperature difference between the surface of the wall and the main body of the fluid in °F.
ΔT_i = inside temperature difference.
ΔT_o = outside temperature difference.

Factors Affecting the Film Coefficient f.—There are several types of heat transfer by convection. In this study we are concerned only with the type in which the fluid does not change phase, i. e., the fluid remains either a liquid or a gas during the process. In this type of heat transfer, the film coefficient depends upon the turbulence of the fluid, increasing as the turbulence increases. The film coefficient also depends upon whether a fluid is being heated or cooled. The difference in the film coefficients for heating and cooling decreases as the turbulence of the fluid increases.

In addition to the various factors already mentioned, the film coefficients are affected by the presence of scale or other deposits on either surface of the pipe wall. Such deposits act as insulation, and may greatly reduce the rate of heat transfer.

Turbulent Flow Inside of a Pipe.—Where a fluid is flowing through a pipe, the film coefficient will increase as the velocity increases. At very low velocities, the flow will be streamline or laminar, and at higher velocities, it is usually turbulent. Also, for natural or free convection, the turbulence and film coefficient increase as the temperature difference ΔT increases.

The critical velocity above which fluid flow in a pipe is turbulent and below which it is streamline can be obtained from:

$$v = \frac{0.58\mu}{D\rho} \qquad (10)$$

where v = velocity in ft./sec.
μ = viscosity in lb./ft./hr.
D = inside pipe diameter in feet.
ρ = density in lb./cu. ft.

Illustrative Problem 3 (Critical velocity for turbulent flow)

Calculate the critical velocity for water in a 3-inch inside diameter pipe. The mean temperature of the water is 130° F. From Table 4, the viscosity of this water is 1.31 lb./ft./hr., and the density is 61.58 lb./cu. ft. Then, from Equation (10)

$$v = \frac{0.58 \times 1.31}{.25 \times 61.58},$$

$$= .049 \text{ ft. per sec. critical velocity.}$$

TABLE 4.
Density and Viscosity of Water

Temperature °F.	Density ρ lb./cu. ft.	Viscosity μ lb./ft./hr.
70	62.31	2.42
80	62.21	2.17
90	62.12	1.96
100	62.00	1.75
110	61.87	1.57
120	61.71	1.43
130	61.58	1.31
140	61.42	1.21
150	61.20	1.11
160	61.03	1.01
170	60.78	.92

Forced Convection Outside of a Pipe

Due to the increased velocity of movement of the liquid past a heat exchanger with forced convection produced by mechanical agitation or stirring, the resistance of the film to the transfer of heat is correspondingly decreased. In general, the decrease in resistance, or in other words, the increase in the film coefficient, is proportional to the velocity of motion past the heat exchanger. Where the agitation of the liquid is irregular, it is very difficult to estimate liquid velocities. But where a steady, uniform stirring action is present, the contents of a circular tank assume a definite motion which depends upon the type and speed of the stirring mechanism.

Positive stirring mechanisms usually found in sludge digestion tanks consist of short horizontal blades mounted on a vertical shaft, as illustrated in Figure 6. The speed of rotation is generally about 60 r.p.m.

The entire liquid mass in a circular tank will not rotate with the same rotary speed as that of the stirrer. The greater the distance from the axis of rotation, the slower will be the rotary motion. It has been determined that the liquid speed of rotation very closely varies as the square root of the diameter of the vertical plane of motion. Thus, the velocity of horizontal motion of the liquid at any distance z from the axis of the stirrer will be

$$v = \frac{\pi\sqrt{2z} \times \text{r.p.m. of stirrer}}{60} \qquad (11)$$

where v = Velocity in ft./sec.
z = Distance in feet from stirrer to vertical plane of liquid motion.

The action of this type of stirring is clearly shown by Figure 6.

Mean Temperature Difference ΔT_m.—The temperature difference between the hot water inside a heating pipe and the cooler water in the digestion tank does not remain constant throughout the length of the heat exchanger. The temperature difference between the two fluids is greatest at the point where the pipe first enters the tank liquid, and smallest where it leaves.

A more exact *mean temperature difference* is a logarithmic function of the two terminal temperature differences. However, for ordinary sludge digestion practice, the arithmetic mean of the terminal temperature difference can be used instead of the logarithmic mean without introducing an error in this value of more than 3 per cent.

Illustrative Problem 4 (*Mean Temperature Difference*)

The average temperature of the liquid in a sludge digestion tank is 85° F. The temperature of the hot water supply to the circular pipe coil is 135° F., and of the return 120° F. Determine the mean temperature difference.

$$\Delta T_m = \frac{(135 - 85) + (120 - 85)}{2}$$

$$= 42.5°F.$$

Determination of Film Coefficients

In the design of sludge digestion tank heat exchangers, we may be concerned with five different types of convection for which film coefficients must be determined, namely

Case 1. Water cooled inside of horizontal or vertical pipes, turbulent flow.

Case 2. Water heated outside of single horizontal pipes, forced convection.

Case 3. Water heated outside of single horizontal pipes, free convection.

Case 4. Water heated outside of vertical banks of piping, free convection.

Case 5. Water heated outside of vertical banks of piping, forced convection.

The same method is used in each case for the determination of the film coefficient. Charts have been prepared which show for each case an arbitrary or assumed coefficient f_b called the "base coefficient." Correction factors are then given for changing these values of f_b to conditions of temperature and pipe diameter other than those assumed for the charts. The general equation for the film coefficient is

$$f_i \text{ or } f_o = f_b \times F_t \times F_d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

Where f_i = inside film coefficient, B.t.u./hr./sq. ft./°F.
f_o = outside film coefficient, B.t.u./hr./sq. ft./°F.
f_b = film coefficient, "base" conditions, B.t.u./hr./sq ft./°F.

Fig. 6. Vertical Stirring in a Sludge Digestion Tank
(*By courtesy of The Dorr Company*)

F_t = temperature correction factor corresponding to the mean temperature T_m of the main body of the water.
F_d = pipe diameter correction factor corresponding to the pipe diameter.

Case 1. *Water Cooled Inside of Horizontal or Vertical Pipes, Turbulent Flow.*—The film coefficient f_i can be determined from the following equation:

$$f_i = f_b \times F_t \times F_d$$

Where f_b = base value of film coefficient, from Fig. 7.
F_t = temperature—correction factor, from Table 5.
F_d = diameter—correction factor for inside diameter of pipe, from Table 6.

TABLE 5.

Temperature Correction Factor for Water—Case 1

Aver. Temp.	F_t
50°F.	0.66
100°F.	1.00
150°F.	1.30
200°F.	1.56

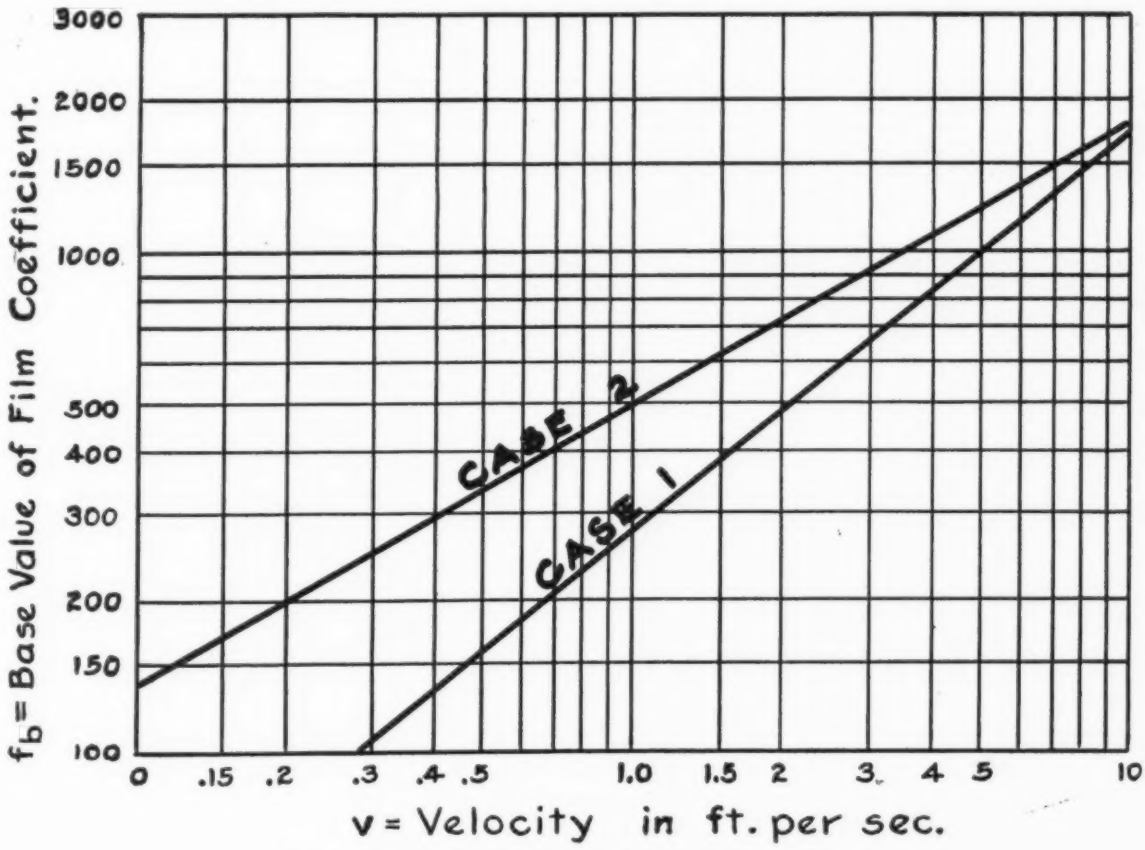

Fig. 7. Base Values of Coefficient f_b for Case 1 *and Case* 2

TABLE 6.
Diameter Correction Factor for Case 1

Inside Pipe Dia. Inches	F_t	Inside Pipe Dia. Inches	F_d
0.20	1.38	1.20	0.97
0.30	1.27	1.30	0.95
0.40	1.20	1.40	0.94
0.50	1.15	1.50	0.92
0.60	1.11	....	....
0.70	1.08	2.00	0.87
0.80	1.05	2.50	0.83
0.90	1.02	3.00	0.80
1.00	1.00	3.50	0.78
1.10	0.98	4.00	0.76

Case 2. Water Heated Outside of Single Horizontal Pipes, Forced Convection.—As previously stated, this type of convection differs from free convection by reason of the artificial movement of the liquid past the pipe caused by mechanical agitation such as stirrers. As in Case 1, the film coefficient f_o can be determined from

$$f_o = f_b \times F_t \times F_d$$

Where f_b = base value of film coefficient, from Fig. 7.
F_t = temperature—correction factor, from Table 7.
F_d = diameter—correction factor for inside diameter of pipe, from Table 6.

TABLE 7.
Temperature Correction Factor for Water—Case 2

Aver. Temp.	F_t
50°F.	0.77
100°F.	1.00
150°F.	1.16
200°F.	1.30

TABLE 8.
Diameter Correction Factor for Case 2

Nominal Pipe Dia. Inches	External Pipe Dia. Inches	F_d
.75	1.050	.98
1.00	1.315	.90
1.25	1.660	.81
1.50	1.900	.75
2.00	2.375	.68
2.50	2.875	.63
3.00	3.500	.58
3.50	4.000	.54
4.00	4.500	.50

Case 3. Water Heated Outside of Single Horizontal Pipes, Free Convection.—In this case the base value of the film coefficient is obtained from Fig. 8 as a function of ΔT_o in °F., in which ΔT_o is the temperature difference between the main body of fluid and the pipe wall. It is sufficiently accurate to take ΔT_o as $\frac{1}{2}\Delta T_m$

$$f_o = f_b \times F_t \times F_d$$

Where f_b = base value of film coefficient, from Fig. 8.
F_t = temperature—correction factor, from Table 9.
F_d = diameter—correction factor for outside pipe diameter, from Table 10.

TABLE 9.
Temperature Correction Factor for Water—Case 3

Aver. Temp.	F_t
50°F.	0.77
100°F.	1.00
150°F.	1.15
200°F.	1.26

TABLE 10.
Diameter Correction Factor for Case 3

Nominal Pipe Dia. Inches	External Pipe Dia. Inches	F_d
.75	1.050	.98
1.00	1.315	.93
1.25	1.660	.88
1.50	1.900	.85
2.00	2.375	.81
2.50	2.875	.77
3.00	3.50	.73
3.50	4.00	.71
4.00	4.50	.69

Case 4. Water Heated Outside of Vertical Banks of Piping, Free Convection.—The method for determining the film coefficient in this case is similar to that given for Case 3, except that no diameter—correction factor is necessary. Thus

$$f_o = f_b \times F_t$$

Where f_b = base value of film coefficient, from Fig. 8.
F_t = temperature—correction factor, from Table 11.

TABLE 11.
Temperature Correction Factor for Water—Case 4

Aver. Temp.	F_t
50°F.	.73
100°F.	1.00
150°F.	1.19
200°F.	1.33

Case 5. Water Heated Outside of Vertical Banks of Piping, Forced Convection.—Approximate values of the film coefficient for water being circulated outside of vertical, parallel-tube heat exchangers by artificial stirring or agitation can be determined as a function of the film coefficients determined for single tubes in Case 2. Thus

$$f_o = 1.2 \times f_{s.t.}$$

in which $f_{s.t.}$ should be evaluated at a velocity as estimated at the narrowest section between the parallel pipes of the unit.

Over-all Heat Transfer.—In the type of heat transfer equipment commonly used in sludge digestion tanks, one fluid receives heat from another fluid, the two fluids being separated by the solid wall of a pipe or tube. When designing such heat exchangers, the temperatures of the fluids inside and outside the pipes are usually known, or are controlling criteria, but the temperature of the pipe wall is not known. Consequently, the expected rate of heat transfer cannot be calculated directly from the film coefficients, since these are based on the temperature difference between the *pipe wall*, and the fluids. However, the expected rate of heat transfer can be readily calculated from the *over-all coefficient U*, because this coefficient is based on the temperature difference between the two fluids. The coefficient U can be defined as the rate of heat transfer from one fluid to another per unit area of the separating wall per degree mean temperature difference between the two fluids. Thus

$$H = AU(\Delta T_m) \quad \text{..... (13)}$$

Where H = total heat transfer in B.t.u. per hour.
U = the over-all coefficient, B.t.u./hr./sq. ft./°F.
ΔT_m = the mean temperature difference between the main bodies of the fluids on each side of the separating on pipe wall in °F.

Relation Between Film Coefficients and Over-all Coefficient.—By the resistance concept, the over-all coefficient U can be calculated from the film coefficients and the conductivity coefficient of the pipe wall when these are known. For thin-walled pipes, the area is based upon the outside surface. Thus, for clean pipes,

$$\frac{1}{U} = \frac{1}{f_i} + \frac{b}{k} + \frac{1}{f_o} \quad \text{..... (14)}$$

Where f_i = film coefficient for the inside of the pipe, B.t.u./hr./sq. ft./°F.
f_o = film coefficient for the outside of the pipe, B.t.u./hr./sq. ft./°F.
f = thickness of pipe wall in inches.
k = coefficient of thermal conductivity of the pipe wall, B.t.u./hr./sq. ft./°F./inch of thickness.

Equation (14) is the theoretical formula for the resistance 1/U to the transfer of heat between the two main bodies of fluid. However, when the values of the coefficients f_i, f_o and k are examined, it can be readily seen that the resistance b/k offered by the pipe wall is most often negligible compared to the resistances $1/f_i$ and $1/f_o$. Therefore, for practical purposes, sufficiently accurate values of the over-all coefficient U can be calculated if the resistance through the pipe wall is omitted. Thus

$$\frac{1}{U} = \frac{1}{f_i} + \frac{1}{f_o} \quad \text{..... (15)}$$

and

$$U = \frac{1}{\frac{1}{f_i} + \frac{1}{f_o}} \quad \text{..... (16)}$$

Effect of Scale on Over-all Coefficient U.—Equations (15) and (16) for calculating the over-all coefficient U involve the assumption that both inner and outer surfaces of the pipe are clean and free from scale. Under sludge digestion tank conditions, the outside of the pipe will always have a crust or scale of appreciable thickness, dependent to some extent on the temperature of the water circulated through the pipe. Higher circulating water temperatures are conducive to a greater degree of incrustation on the pipes. Therefore, it is good operating practice to keep the temperature of the hot water supply to the heat exchanger below 150° F.

The inside of a heating pipe will also accumulate some scale. The degree of scale is almost entirely dependent upon the hardness of the water. A hard water is conducive to scale formation while a soft water will be correspondingly free from this trouble.

The following values for dirt and scale coefficients have been taken from the Standards of Heat Exchange Institute[9].

TABLE 12.
Scale Coefficients "S" in B.t.u./hr./sq. ft./°F.
S = Scale Coefficient

Kind of Water	125°F. or less	Over 125° F.
Hard Water (over 15 grains/gal.)	330	200
Lake and Eastern Well Waters	1000	500
River Waters		
Maximum	500	330
Mississippi	330	250
Delaware, Schuylkill	330	250
East River	330	250
Chicago Sanitary Canal	130	100
Sewage Sludge	70	50
Distilled Water	2000	2000

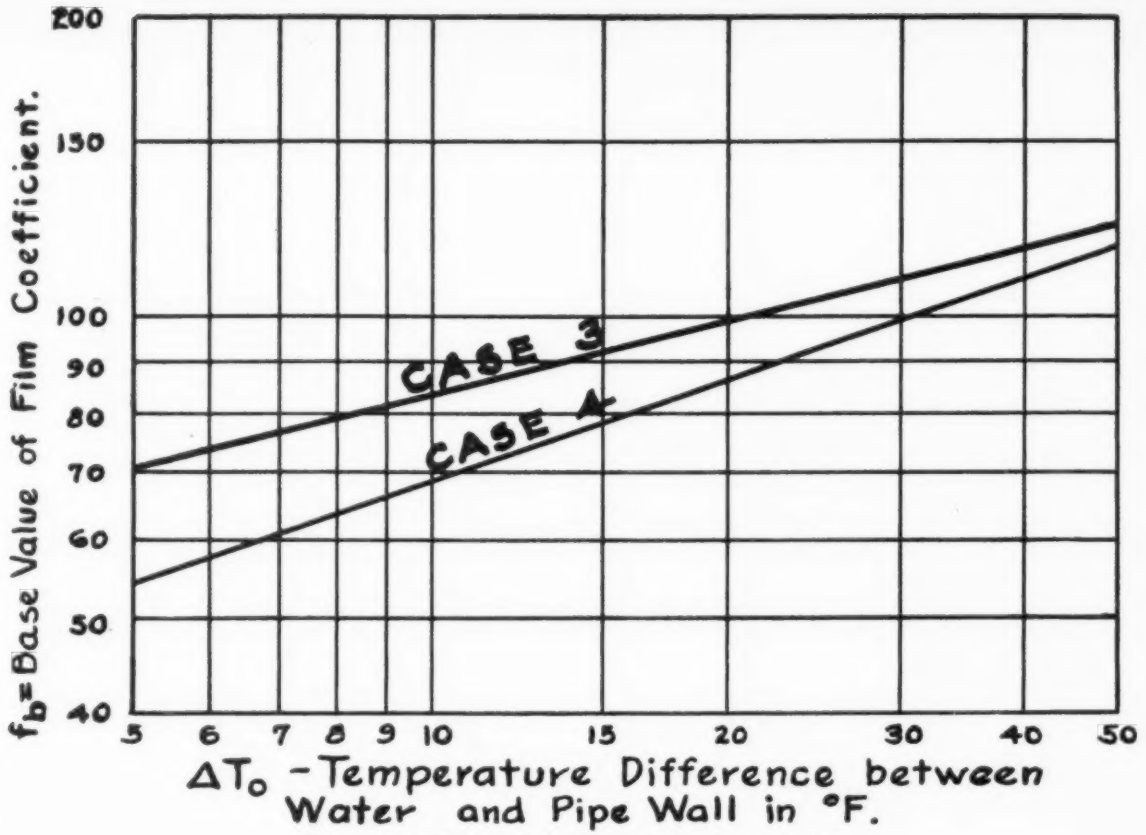

Fig. 8. Base Values of Coefficient f_b for Cases 3 and 4

Therefore, if the resistances due to inside and outside scale formations are added to Equations (15) and (16), then for thin-walled pipes

$$\frac{1}{U} = \frac{1}{f_i} + \frac{1}{f_o} + \frac{1}{S_i} + \frac{1}{S_o}$$

$$U = \frac{1}{\frac{1}{f_i} + \frac{1}{f_o} + \frac{1}{S_i} + \frac{1}{S_o}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

where S_i = inside scale coefficient, B.t.u./hr./sq. ft./°F.
S_o = outside scale coefficient, B.t.u./hr./sq. ft./°F.

Circulating Water.—The quantity of hot water to be circulated through a heat exchanger depends upon the total heat to be transmitted, and the temperature drop between the hot water supply and return. Obviously, the greater the temperature drop for a given B.t.u. exchange, the lower is the quantity of hot water required, and vice versa. A practical temperature drop is about 20° F., as for example, if the hot water supply is 130° F., the return will be 110° F. Therefore, the quantity of water required is

$$Q = \frac{\text{B.t.u./hr.}}{(T_s - T_r) \times 60 \times 8.33 \text{ B.t.u./hr.}} = \frac{}{(T_s - T_r) \times 500} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

where Q = rate of hot water circulation, g.p.m.
T_s = temperature of hot water supply, °F.
T_r = temperature of hot water return, °F.

Illustrative Problem 5 (*Design of Heat Exchanger—Horizontal Heating Pipes with Free Convection*)

The sludge digestion tank shown in Figure 2 is to be provided with heat by means of a spiral-form horizontal heat exchanger. This problem is a continuation of Problems 1 and 2, the three problems comprising a comprehensive solution of a sludge digestion heating application. Pertinent data are as follows:

Heat required ..... = 256,900 B.t.u./hr.
Tank Floor to bottom pipe = 6 feet
Vertical spacing of pipes.. = 12 inches
Mechanical Agitation ..... = None
Convection ..... = Free
Temperature of Sludge.... = 85°F.
Temperature H.W. Supply. = 130°F.
Temperature H.W. Return = 110°F.
Circulating Water ..... = Well water, 100 ppm hardness

From Equation (18)

$$Q = \frac{256{,}900}{(130 - 110) \times 500} = 26 \text{ g.p.m.}$$

Since the base coefficient f_b for the inside film depends upon the velocity of flow through the pipe, it is necessary to assume a pipe size which appears in advance to fit the conditions. If the heat exchanger based upon this pipe size is found to be unbalanced, a second calculation must be made for another pipe size. If a 2½ inch internal diameter, wrought iron pipe is assumed, then the velocity of the hot water through the pipe is 1.68 ft./sec. Since this is greater than .049 ft./sec., the flow is turbulent (see Equation 10).

The inside film coefficient f_i can be obtained from Case I, and the outside film coefficient f_o from Case 3. From Figure 7, $f_b = 410$ B.t.u./hr./sq. ft./°F.; from Table 5 by interpolation, F_t for 120° F. = 1.12; from Table 6, F_d for 2½ inch pipe = 0.83. Hence,

$$f_i = 410 \times 1.12 \times 0.83 = 382 \text{ B.t.u./hr./sq. ft./°F.}$$

The average temperature difference ΔT_o, inside to outside water, is $(120 - 85)/2 = 17.5°$ F. From Figure 8, $f_b = 95$ B.t.u./hr.; from Table 9 by interpolation, F_t for 85° F. = 0.86; from Table 10, F_d for 2½ in. pipe = 0.77. Hence

$$f_o = 95 \times 0.86 \times 0.77 = 63 \text{ B.t.u./hr./sq. ft./°F.}$$

The inside and outside scale coefficients can be obtained from Table 13.

S_i = 500 B.t.u./rr./sq. ft./°F.
S_o = 70 B.t.u./hr./sq. ft./°F.

Thus in Equation (17),

$$U = \frac{1}{\frac{1}{382} + \frac{1}{63} + \frac{1}{500} + \frac{1}{70}} = 28.8 \text{ B.t.u./hr./sq. ft./°F.}$$

The total effective area of heating pipe surface required to transmit 256,900 B.t.u./hr. can be obtained from Equation (13).

$$H = AU(\Delta T_m)$$

and

$$A = \frac{H}{U(\Delta T_m)} = \frac{256{,}900}{28.8 \times (120 - 85)} = 255 \text{ sq. ft.}$$

The external area of 2½ inch wrought iron pipe is .753 sq. ft.

per foot of length. The minimum length of heating pipe required will be 255/.753 = 338 lineal feet.

One spiral pipe coil of 39 ft. diameter (1 ft. less than the inside diameter of the sludge digestion tank) has a length of 123 lineal feet. Thus, the minimum number of coils required is 338/123 = 2.8.

A heat exchanger consisting of 3 — 39-ft. diameter coils of 2½ inch wrought iron pipe will satisfy the requirements of the problem if 26 g.p.m. of 130° F. water is circulated continuously.

Illustrative Problem 6 (*Design of Heat Exchanger—Vertical Bank Type of Unit with Forced Convection*)

The sludge digestion tank shown in Figure 4 is to be provided with heat by means of a vertical-bank type of heating unit. The sludge will be stirred by a vertical stirrer with stirring blades as shown in Figure 6. Pertinent data are as follows:

Heat required	= 256,900 B.t.u./hr.
Spacing of pipes	= 6 inches
Mechanical agitation	= Rotary Stirrer, 60 r.p.m.
Convection	= Forced
Temperature of Sludge	= 85°F.
Temperature of H.W. Supply	= 130°F.
Temperature of H.W. Return	= 110°F.
Circulating Water	= Well water, 100 ppm hardness

From equation (18)

$$Q = \frac{256{,}900}{(130 - 110) \times 500} = 26 \text{ g.p.m.}$$

If a 2½ inch internal diameter pipe is assumed as in Problem 5, the velocity of the circulating water will be 1.68 ft./sec. The inside film coefficient f_i can be obtained from Case 1, and the outside film coefficient f_o from Case 5.

From Figure 7, $f_b = 410$ B.t.u./hr./sq. ft./° F.; from Table 5 by interpolation, F_t for 120° F. = 1.12; from Table 6, F_d for 2½ inch pipe = 0.83. Hence

$$f_i = 410 \times 1.12 \times 0.83 = 382 \text{ B.t.u./hr./sq. ft./°F.}$$

The distance from the stirrer shaft to the center of the vertical heat exchanger is 5 feet. Since the speed of the stirrer is 60 r.p.m., the peripheral velocity of the liquid through the heat exchanger will be from Equation (11)

$$V = \frac{\pi \sqrt{10} \times 60 \text{ r.p.m}}{60} = 10 \text{ ft./sec.}$$

From Figure 7 (Case 2), $f_b = 1800$ B.t.u./hr./sq. ft./°F.; from Table 7 by interpolation, F_t for 120° F. = 1.06; from Table 8, F_d for 2½ inch pipe = .63. Hence, from Case 5

$$f_o = 1.2 \times f_{s.t.} = 1.2 \times 1800 \times 1.06 \times 63 = 1450 \text{ B.t.u./hr./sq. ft./°F.}$$

The inside and outside scale coefficients can be obtained from Table 13.

$S_i = 500$ B.t.u./hr./sq. ft./°F.
$S_o = 70$ B.t.u./hr./sq. ft./°F.

Thus, in Equation (17)

$$U = \frac{1}{\frac{1}{382} + \frac{1}{1450} + \frac{1}{500} + \frac{1}{70}} = 56.0 \text{ Bt.u./hr./sq. ft./°F.}$$

The total effective area of heating pipe surface required to transmit 256,900 B.t.u./hr. can be obtained from Equation (13)

$$H = AU(\triangle T_m)$$

and

$$A = \frac{H}{U(\triangle T_m)} = \frac{256{,}900}{56.0 \times (120 - 85)} = 131 \text{ sq. ft.}$$

The external area of 2½ inch wrought iron pipe is 0.753 sq. ft. per foot of length. The minimum length of heating pipe required will be 131/.753 = 174 lineal feet.

Each vertical pipe in the bank-type heat exchanger has 25 feet of its length immersed in the liquid to be heated. The horizontal hot water supply and return pipes will have a minimum length of 20 feet from the heat exchanger to the tank wall. Thus, the total length of effective 2½ inch piping, if six vertical pipes are used in the heat exchanger, will be

Heat exchanger—25 × 6	= 150 lin. ft.
H. W. supply and return—20 × 2	= 40 lin. ft.
Total effective length	= 170 lin. ft.

This heat exchange piping will meet the requirements of the problem if 26 g.p.m. of 130° F. water is circulated continuously.

SELECTION OF HEATING EQUIPMENT

Standard equipment and control devices for heating hot water are too well known to warrant much discussion here. The usual sources of heat are conventional coal, oil, or gas-fired hot water boilers. A great majority of municipal sewage treatment plants use one or more of these. However, in many industrial plants, low-cost steam or electric current is available, and in such cases it is usually much more economical to use these sources of heat than it would be to recover the gases of sludge digestion for heating purposes, or to use any auxiliary fuel.

Where steam is used as a prime source of heat, the digestion tank circulating water can be heated automatically to any desired temperature by means of an instantaneous type of shell and tube hot water heater. With this type of heater the steam is introduced in the shell, and the hot water is circulated through the tubes. The quantity of steam admitted into the heater is controlled by a thermostatically-operated steam valve.

Where electric current is used as a prime source of heat, large capacity electric circulation heaters are used. These heaters have limited storage capacity, and depend entirely upon the heating element to be of a size sufficient to heat the water as it is circulated through the heater. A thermostatically-operated switch controls the amount of electric current applied in order to maintain the hot water within a predetermined temperature range.

Hot water circulating pumps are almost mandatory in order to insure a positive circulation of the proper quantity of hot water through the digestion tank heat exchanger at all times. Such circulating pumps may be manually controlled, but it is preferable to operate them automatically by means of a thermostatic switch of the Minneapolis-Honeywell or Mercoid Corporation type. The actuating temperature in this arrangement must be the temperature of the digestion tank liquid which is being heated.

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*Added by the editors. An article published since author's preparation of this paper.

Ed. Note: Readers are referred to the Consolidated Index, which lists other articles on sludge heating.

EVALUATION OF METHODS OF HEATING SLUDGE*

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THE application of heat to sludge before or during the digestion process is complicated by several factors peculiar to the material handled and the treatment to which it is subjected. In order to evaluate properly the various methods of supplying heat it is essential that these factors be considered and the limitations which they impose be recognized.

Factors Peculiar to Sludge Digestion

The heat requirements vary at different stages of the digestion process. A considerable amount of heat is required to raise the incoming sludge to the temperature of the digestion tank. As the digestion proceeds, it is essential that the temperature be maintained within a certain optimum range. As digestion nears completion, it is less important to maintain the sludge within this range.

The gas formed by the decomposition of sludge forms a highly explosive substance when mixed with the proper amount of air. While good design practice would practically eliminate the possibility of air entering a digester, it is always wise to be on the safe side and not use any type of heating equipment which would make possible the introduction into a digester of an open flame or a hot surface approaching ignition temperature.

The sludge contains millions of living organisms essential to the digestion process, but which can be destroyed by heat. It is therefore essential that no method of introducing heat into the sludge be used which would raise too great a proportion of the sludge to excessive temperatures for prolonged periods of time.

Basic Principles of Heat Exchange

In practically all methods in common use for heating sludge, the heat is supplied by conduction and convection. Some metal surface (heat exchanger) is usually heated directly or by a hot liquid and this surface transfers its heat to the adjacent sludge. The transfer of heat through the metal wall is by conduction and through the sludge by a combination of convection and conduction.

Between the metal surface and the main body of the sludge, however, is a thin film of sludge through which heat is transferred solely by conduction. Also on either or both sides of the metal wall there may be a scale which further inhibits the transfer of heat. It can therefore be seen that the transfer of heat from a metal wall to the sludge is a function of the conductivity of the metal wall; the conductivity of the scale, if present (known as scale coefficient); the conductivity of the intermediate film (known as film coefficient); and the conductivity of the sludge.† Finally the quantity of heat required to raise the temperature of the sludge to a given level is a function of the specific heat of the sludge, which varies with the type, concentration, and degree of digestion. Any system of applying heat to sludge would have to be evaluated in the light of its effect on one or more of the various factors just mentioned, or the elimination of some of them.

The numerical values of the thermal conductivity of heat exchanger walls and scale coefficients for both hot and cold sides of heat exchangers have been determined and are available in various books on heat exchangers. Since the specific heat and conductivity of sludges, and film coefficients for the sludge side of heat exchangers are not nearly so readily available, however, some values will be presented here as a basis for future discussion.

Specific Heat

The specific heat of sludge is usually taken as unity. Sierp[11], using the method of mixtures, concluded that the specific heat of sludge is practically the same as that of water.

Conductivity

Very little information is available on this subject. Keefer and Kratz[8] give the following values as determined for them on certain samples by the Bureau of Standards:

	Moisture (%)	Volatile Matter (%)	Thermal Conductivity (B.t.u./hr./sq. ft./deg. F./in.)
Raw sludge	87.4	70.8	4.5
Semi-digested sludge	87.4	65.1	5.1
Digested sludge.	87.3	51.1	4.7
Water (for comparison)	...	...	4.1

Film Coefficient

A few scattered figures are available for the common hot water heating coils in digestion tanks. P. Langdon[5], reviewing the data from eight plants, states that values range from 10 to 20 Btu. per hr. per sq. ft. per deg. F. Queer[9] in laboratory studies using ¾-in. brass pipe obtained values ranging from 12 for heavy sludge to 88 for supernatant. The value for water in the same apparatus he determined as 116. Walraven[12] from field tests made in a digester with new clean heating coils obtained 11 to 12 for sludge with 12 per cent solids and 39 for strong supernatant. Young and Phillips[14] using a 2-in. wrought iron pipe coil and sludge between 5 and 12 per cent solids obtained 17.7. For heat exchangers outside the digester and using two concentric pipes, the inner carrying sludge and the outer hot water, P. Langdon[6] estimates values to be from 200 to 500 Btu. per hr. per sq. ft. per deg. F.

The author is at present engaged in making laboratory and field tests for securing further data on specific heats, conductivities, and film coefficients for both natural and forced convection.

As seen from the data presented, sludge has about the same specific heat as water, a slightly greater conductivity, and a very low film coefficient. Therefore the problem of heating sludge as compared to that of heating water resolves itself largely into methods of overcoming the sludge film coefficient bottleneck.

The methods used in modern sanitary engineering practice for heating sludge digestion tanks can be divided into two general classifications, namely, those which introduce heat directly into the digester and those which heat the sludge outside the digestion tank. Each of these types will be considered briefly and an attempt made to evaluate their suitability.

Heaters Within the Digester

Horizontal Hot Water Coils

This is by far the most common method in use, although newer methods are rapidly challenging its previous almost complete monopoly of the field. The heating coil is in the form of a spiral around the inside periphery of the tank. Hot water is admitted at the top of the coil and removed at the bottom. The coils should be placed in the most rapidly digesting sludge, which in single stage tanks is usually considered to be in the zone between a few feet above the bottom to slightly over mid-depth.

The main disadvantage of this method is the absolute dependence for heat transfer on the natural convection currents induced in the sludge by the heating coils and the circulation caused by gas bubbles rising to the surface. In many digestion tanks, slow moving mixtures are used, but these are primarily for the purpose of insuring more uniform gas production. As already mentioned, the film coefficient of heat transfer from the coils to the sludge is very low, about one-tenth that for water. Bacon[2] recommends placing the coils at least 3 ft. from the side walls to promote better convection currents.

When it is also realized that because of the tendency for a highly insulative scale to form on the outside of the coils unless the temperature of the heating water is kept below 130° F., thereby making a very small differential temperature between the inside and outside of the heating coil, the disadvantage of a low film coefficient is apparent.

Another important disadvantage of this method of heating is the difficulty of making repairs to the coils. As pointed out by L. Langdon[7], failure of a coil means that the digestion tank must be taken out of service for a considerable period of time, as it must be completely cleaned and ventilated before workmen can enter because of explosion hazards. As he further points out, in many plants with the digester out of service the entire plant must be bypassed as there is no other place to put the sludge. Failure of a coil has in some instances caused sludge to be drawn into the heating boiler or a gas engine where engine jacket water is circulated through the coils.

Vertical Hot Water Pipes

In an effort to correct some of the aforementioned difficulties, some plants have installed vertical heating pipes. As stated by Wittwer[13], this type of heater should be removable from the top. The vertical pipes are in series and are placed from 5 to 7 inches on centers. Hot water is introduced into the top of the first pipe and is circulated down and up through succeeding pipes until removed from the top of the last pipe. This type of heater is placed at or near the center of a small digester, and several are distributed uniformly about larger tanks. In some installations the

*From *Sew. Wks. Jour.*, March, 1947. Reprinted by permission of the Fed. Sew. Wks. Assns.

†See Appendix.

vertical circulation of the sludge is aided by impellers rotating about a vertical axis.

The vertical pipes have the decided advantage of ease of removal of the entire heater or individual pipes for cleaning. Also there is less tendency for scum to adhere to the outside pipe wall, which in the case of horizontal coils sometimes results in failure of the pipe supports. Unless aided by artificial circulation it is questionable if the film coefficient of heat transfer is greater than that of horizontal coils.

Submerged Gas Burners

In a recent article, Keefer[8] describes two new types of heaters in which the heat of a gas flame is transmitted direct to the inside of a heat exchanger submerged in the digestion tank:

"The open type burner consists of a short length of pipe of any desired diameter, open at the discharge end. Near the open end is a perforated plate, through which the air-gas mixture flows. . . . The advantage of this type of burner is that the flame and the burnt gases make direct contact with the sludge, the result being that there is practically no heat loss. The closed type burner is designed so that the flame and products of combustion are contained within a closed container, constructed of pipe and immersed in the sludge."

Keefer claims that in a two months' test of an open type heater no insulating crust was formed on the outside of the heater walls. No statement is made by him regarding the closed type heater, but it is assumed that it is expected to behave in the same way, otherwise it would have to be removed at frequent intervals for cleaning. The closed type heater has the definite advantage over the usual hot water coil in that a much greater temperature differential across the walls of the heat exchanger can be used, thereby increasing the rate of heat transfer although the film coefficient remains the same. The open type heater has the additional advantage of increased circulation of the sludge past the outside of the heater due to the discharged gases, thereby increasing the film coefficient of heat transfer. Discussing the advisability of introducing burnt gases directly into the digester, thereby diluting the digester gas which is to be used as fuel, Keefer states that "designs have also been prepared which would provide for an open type burner in a covered digester and which would eliminate the possibility of burned gases mixing with the unburned gases." The writer invites discussion on the advisability of introducing an open flame into a digester, a procedure which radically differs from present safety practices.

"Radiant" Heating

A new method proposed by Wenzel[1] is that of applying "radiant" heating to digesters. Since exposed heating pipes are a source of so much trouble, he suggests burying the coils in the concrete floor and walls. Since the tank would be full of sludge, this would not really be a case of radiant heating but of conduction and convection, the heat exchanger being the tank itself. The film coefficient should be practically the same as that for the wrought iron heating coils, but in addition to the conductivity of the pipe walls, which is usually taken at about 370 B.t.u. per hr. per sq. ft. per deg. F. per inch thickness, there would be that of the concrete which is about 12 units. Thus, in effect, the added resistance of the concrete would inhibit the flow of heat.

Steam and Hot Water

Other methods for heating sludge in digestion tanks include the direct introduction of steam or hot water into the tanks. The use of steam, proposed by Rawn[10], will be discussed later under outside heat exchangers. Commenting on the use of hot water, Wittwer[13] states that it is of German origin and up to the present time has not found much favor in this country. He mentions studies by Keefer[4], who found that more heat was required to heat the sludge by the direct addition of hot water than was the case with well designed heat exchangers using circulated water. The cost of the pipe heating coils is saved but water must be continually purchased and cannot be reused. He also mentions studies by Downes[3], who claims that the practice of introducing hot water directly into the sludge tends to segregate grit and coarse particles. The continual addition of hot water also takes up valuable space in the digester or adds to the volume of supernatant to be treated.

Heaters Outside the Digester

Addition of Steam to Raw Sludge

Rawn[10] and Bacon[2] have successfully used live steam to heat the daily additions of raw sludge to the digester, thereby maintaining the desired temperature in the tanks. The raw sludge is transferred from the sedimentation tanks to sludge concentration tanks, where it is mixed with 20 per cent by volume of digested sludge for "seed." The mixture is then transferred to the sludge digestion tank and, as the transfer pump starts, live steam is directed into the hopper bottom of the concentration tank from which the sludge mixture is drawn. The piping arrangement is such that preheating and recirculation to the concentration tanks is possible. As the digesters show a temperature drop of only 2 or 3° F., no further heating of the sludge is necessary. The steam pressure is less than 15 p.s.i.

In answer to the question as to the quantity of steam condensate added to the sludge, Bacon[2] states "neglecting losses this amounts to . . . 1 gallon per 10° F. temperature rise for every 110 gallons of sludge heated."

The advantage claimed by Rawn[10] for this method of heating sludge as compared to other types of heat exchangers outside the digestion tank is its flexibility, permitting the use as a heat exchanger, of any receptacle which will hold sludge, and making possible easy and quick adjustments for variation in the temperature of the incoming sludge, by regulating the flow of steam or the speed of the transfer pump. This method has the further advantage of eliminating entirely the sludge film coefficient of heat transfer bottleneck, that is encountered in all other methods already mentioned. The question of sterilization of the sludge by the live steam is answered by the successful use of the method for over six years. Apparently the number of bacteria killed by the steam is very small in proportion to the total number present.

Hot Water Heat Exchangers

L. Langdon[7] and P. Langdon[8] describe heat exchangers in which the sludge is circulated through pipes surrounded by hot water. The former type has the heating coils immersed in a hot water bath. A pump continuously circulates the sludge in the upper two-thirds of the digester through the pipes in the heat exchanger. Provision is made for adding the raw sludge to the circulating material before it enters the heater. This arrangement is particularly recommended for smaller plants where sludge additions to the digester are intermittent and preheating the raw sludge on a batch basis would require excessive heating capacity which would operate intermittently and with considerable manual attention.

The latter type developed for the Hunts Point Plant in New York City consists of two concentric aluminum pipes, 6 and 8 inches in diameter. The raw sludge is pumped through the inner pipe and hot water circulated in the opposite direction through the annular space. It is estimated that under the most severe weather conditions the sludge will have to be heated to 97° F. in the exchangers in order to maintain a temperature of 85° F. in the digesters.

The choice of aluminum was based primarily on corrosion resisting properties. At maximum load the water passing through the heat exchanger will drop from 138° F. to 118° F. and the sludge will be raised from 50° to 108° F. It is expected that the temperature will be high enough to melt any grease which forms on the inside of the sludge pipes but low enough to avoid sludge caking.

One decided advantage of the forced circulation heat exchangers over the natural convection type is that the sludge film coefficient of heat transfer is increased. The only data available to the author is the estimate of P. Langdon[6] for the Hunts Point Plant of 200 to 500 B.t.u. per hr. per sq. ft. per deg. F.

Comparison of Sludge Heating Methods

It is apparent that external heat exchangers have a definite advantage over the internal heating coils in the amount of heat that can be transferred to the sludge per square foot of exchanger surface. This might counter-balance the objection raised to outside heat exchangers that the sludge must be raised to a higher temperature to overcome all the losses from the exchanger and digester.

One decided advantage of the external type of heater is its accessibility for cleaning and repair. Piping can be arranged so that different exchangers can be used with the same digestion tank, thereby avoiding the necessity of taking a digester out of service when making repairs on the heater, as is the case when each digester is dependent entirely upon its own internal heating system. External heaters also make it possible as pointed out by Rawn[10] to "control the sludge temperature in any receptacle which will hold sludge, whether it be an earthen pit, a lined retention chamber—covered or uncovered—an unused tank or basin, or a regulation sludge digestion unit."

Finally, as mentioned by L. Langdon[7] the external heat exchanger avoids the sudden temperature drops and accompanying decreased gas production noticed in some digesters when unheated raw solids are added.

Appendix

Mathematical relationship of heat transfer coefficients:

$$q = UA(T_1 - T_0)$$

$$\frac{1}{U} = \frac{1}{h^1} + \frac{1}{hd^1} + \frac{x}{k} + \frac{1}{hd^{11}} + \frac{1}{h^{11}}$$

where q = rate of heat transfer, B.t.u. per hr.

U = overall coefficient of heat transfer in B.t.u. per hr. per sq. ft. per deg. F.

A = area of heat transfer surface in sq. ft.

T_t = Bulk temperature of warmer fluid, degrees F.

T_s = Bulk temperature of cooler fluid (sludge), degrees F.

h^1 = film coefficient between hot liquid and hot surface of heat exchanger in B.t.u. per hr. per sq. ft. per deg. F.

h_d^1 = scale coefficient on hot side of heat exchanger in B.t.u. per hr. per sq. ft. per deg. F.

h_d^{11} = scale coefficient on cool side of heat exchanger in B.t.u. per hr. per sq. ft. per deg. F.

h^{11} = film coefficient between cool surface of heat exchanger and sludge in B.t.u. per hr. per sq. ft. per deg. F.

k = thermal conductivity of heat exchanger wall in B.t.u. per hr. per sq. ft. per deg. F. per ft. thickness

x = thickness of wall of heat exchanger in feet.

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Discussion

By WILLEM RUDOLFS

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Although Mr. Miles has not taken into consideration the question of heat losses from digestion tanks, such losses have some bearing on the methods of heating employed. Incompletely insulated or poorly protected tanks require a greater heat supply and are subject to greater fluctuations in heat losses than well insulated tanks. With the possible use of higher temperatures in the range of thermophillic digestion heat losses increase. When preheating methods of sludge are utilized the question of preventing heat losses is even more important.

Laboratory experiments made some years ago showed that the rate of heat loss from sludge varied directly as the difference in temperature between the heated material and the surrounding air. The rate of heat loss was practically the same from fresh solids and ripe sludge. No difference in the rate of heat losses from sludges ranging in concentrations from 4 to 11 per cent were observed.

Large scale experiments with an operating digestion tank 30 ft. in diameter and 17 ft. deep, insulated in the usual way by an earthen embankment and sludge heated to about 87° F., showed that of the total heat supplied, 70 per cent was utilized and 30 per cent lost. The heat required to maintain the digestion tank at a given temperature varied directly as the temperature difference between the tank and the normal ground temperature.

The heat from the digestion tank did not penetrate the same distance in all directions into the surrounding soil, probably due to wind action and soil drainage. The heat from the sides of the tank travelled only in a horizontal direction. Preheating of sludge outside the digester must be materially higher than the actively digesting material.

Mr. Miles indicated that stirring devices are utilized primarily to induce more uniform gas production. In effect, such stirring devices are usually located near the bottom of the tank and aid materially in the concentration of the sludge and may be of some benefit in increasing convection currents and thus aiding in the heat distribution throughout the mass.

Normally the contents of the digestion tank consist of two distinct layers of different solids concentrations and different temperatures. The temperatures within the layers are practically constant horizontally and vertically. The difference in temperature between the layers averaged about 10° F. over a period of one year of careful measurement, the hotter layer being toward the bottom. It appears, therefore, that center heating devices do not need to be rotating in order to induce convection currents as an aid for heat distribution.

The film coefficient of heat transfer from the testing coils to the sludge is very low, about 1/10 that of water. It should be remembered that the cooling constant of sludge is also much lower than water, in other words, the sludge retains the heat longer and to a higher degree than water, which in turn aids in maintenance of a uniform distribution of heat no matter which heating device is employed.

Mr. Miles stresses the tendency for a highly insulative scale to form on the outside of the coils unless the temperature of the testing water is kept below 130° F. This recalls a series of simple laboratory experiments made about 1927 and which were never published except as a commentary statement that scale formation is rapid when sludge is heated to 140° F. and to be on the safe side the incoming hot water should not be over 135° F.

The writer does not know of any experiments which have confirmed these simple laboratory experiments nor has he seen a compilation of heating water temperatures and rates and quantities of scale formation from operating plants. Apparently practice has taken this statement and found that no undue scale formation occurs with temperatures below 130° F. Further study may be warranted to establish more precisely the temperature limits. Such a study would be of particular interest in connection with the submerged gas burners and preheating devices.

Attention should be called to the fact that addition of steam to raw sludge prior to introduction into the digesters, as well as preheating of sludge through steam and hot water jacketed bundles of pipes, have been practiced at various places for a number of years.

From a comparison of sludge heating methods Mr. Miles appears to favor the external type of heater over the internal heating coils. It may be expected that the designing engineers will scrutinize the results obtained rather than act upon expectations, but there is no doubt that the heating methods most commonly employed can be improved, modified or varied to advantage and that newer methods are worthy of careful study.

Discussion

By A. M. RAWN, VINTON W. BACON and HARVEY F. LUDWIG

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At Prof. Mile's invitation, we have prepared this brief discussion of sludge heating with live steam, with particular reference to operating experience with this method at the Joint Disposal Plant of the Los Angeles County Sanitation Districts.

As described by Prof. Miles, the sludge is heated at the Districts' plant by injecting live steam into the hopper of the sludge concentration tanks while the raw sludge is being withdrawn and pumped to the digesters. Very little heat is lost from the digestion tanks, and no additional heating is required. This system has been continuously in operation since 1942 and has proven easy to operate and practically free of trouble. The one minor difficulty encountered resulted from injecting the steam through a 3-in. open-end pipe; this caused considerable "booming" noise and also cracking of the concrete floor immediately below the pipe outlet. This difficulty, however, is easily overcome by use of multiple steam outlets of small size.

Steam heating of sludge has also been successfully practiced for more than a year at the San Diego, Calif., sewage treatment plant, according to Supt. Eric V. Quartly. In this instance a 6 ft. by 6 ft. by 6 ft. concrete box, open at the top, was constructed on the roof of the control house between the primary and secondary digestion tanks. An overflow weir maintains a 5 ft. sludge depth in the box, and raw sludge is added through a single pipe in the center. Steam is supplied through a 4-in. transmission line into a header comprising four 3-in. pipes. These 3-in. pipes are sealed at the ends but have been perforated near the ends, with eight 3/16-in. holes in each pipe. This multiple small-sized outlet system has resulted in operation free of noise and vibration.

Steam is generated in a 75 h.p. Scotch marine boiler, and a pressure of only 3 psi. is sufficient to overcome transmission line losses and the static head of the sludge. No further sludge heating is needed at the digesters, temperatures of which are maintained throughout the year between 96° and 99° F. The raw sludge is pumped almost continuously from the clarifiers to the digesters so that steam may be added continuously and in constant amount. (Such steady operation might not be practical in small plants.)

There are other methods for heating sludge with steam. In closed sludge transfer system, where sludge is pumped directly from sedimentation tanks to digesters, steam may be injected into the pump suction lines, with the advantage that only low-pressure or heating-class boilers would be necessary. When steam is injected into pump discharge lines, higher pressures are of course required, with the result that power-class boilers may have to be installed. Only in larger plants would high-pressure systems be economical, but in these cases good design might take advantage of the pumping power of the steam jet.

Satisfactory digester temperatures can also be maintained by injecting the steam through a few outlets directly into the digester. This scheme should be particularly suited to small plants, because it permits continuous steam injection and therefore smaller capacity boilers. It is the writers' belief that no appreciable sterilization effects can result from such injection, since the areas which might actually be sterilizing temperatures are relatively very small. The volume of sludge which might be sterilized is negligible, and will be quickly reseeded by the remaining mass. In the experimental work preceding installation of the Los Angeles County Sanitation Districts' steam system, Dr. Richard Pomeroy found that no appreciable sterilization resulted from direct injection.

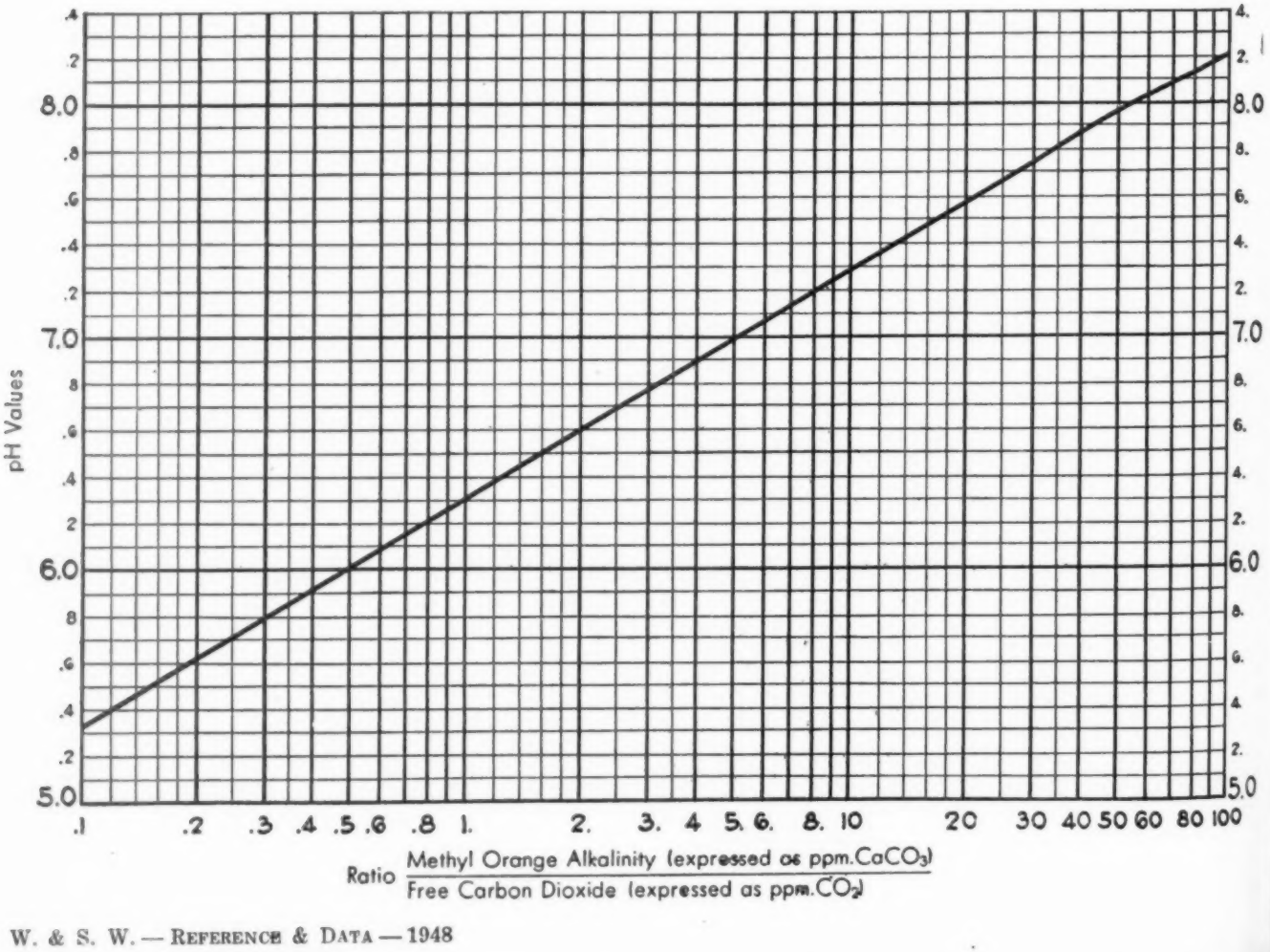
The thermodynamics involved in the various methods of steam injection are simple. The heat exchange is direct from steam to sludge, without the complications that arise from use of heat exchangers. The problems of conductance through the metal walls of an exchanger, through the scale which inevitably forms upon the walls, and through the films adjacent to the scale or wall do not exist.

There does not exist loss of heat in the present type of steam heating systems at the heat generating plant through boiler stacks, and a small amount in steam transmission lines, but these losses are common to any system where the processes of heat generation and dissipation are separated or where hot flue gases are allowed to escape.

The most efficient heating system will be that in which the heat is generated and dissipated within the sludge mass, and for this reason the recently advanced idea of burning a fuel in direct contact with the sludge (the submerged gas burner with open flame described by Keefer) offers considerable promise. At the present time much work remains to be done before this idea can safely be incorporated into plant design.

RELATIONSHIP BETWEEN ALKALINITY CARBON DIOXIDE AND PH VALUES

(Courtesy of the Permutit Co.)


pH Values
8.0
7.0
6.0
5.0
.1 .2 .3 .4 .5 .6 .8 1. 2. 3. 4 5. 6. 8. 10 20 30 40 50 60 80 100
Ratio Methyl Orange Alkalinity (expressed as ppm.CaCO3) / Free Carbon Dioxide (expressed as ppm.CO2)

SCUM CONTROL IN DIGESTERS*

By R. S. RANKIN

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SCUM formation in sludge digesters is a serious problem in a certain percentage of plants. In a larger percentage this problem either is not present, or the operator has learned how to keep it under control.

Scum may be considered a problem in a digester when it interferes with uniform gas production, when it plugs pipe outlets and stops the normal overflow, when it interferes with the withdrawal of tank contents, or when its volume is so great as to reduce materially the available digestion capacity.

In some plants scum may attain a depth of several feet occupying as much as 30 per cent of available capacity and resisting all reasonable efforts to dissipate it. Persistent accumulations, if allowed to remain in a digester, can occasionally do positive damage. For example, when the scum has sealed off or plugged the normal overflow lines, continued pumping of sludge may force dislodgement of the tank cover, as has actually occurred in a few installations. Also, when a scum-laden tank is emptied, the layer may have become so compact that, as it drops with the receding level, it pulls down with it the gas and even the hot water piping with all their supports.

The question naturally arises as to why some plants are continually faced with this problem, and other plants seldom, if ever, experience it. The problem under discussion, incidentally, refers to persistent scum in the digester after routine operation has been established, and not to a temporary foaming condition sometimes occurring during early stages of operation.

Causes of Scum Formation

It is known that some types of sewage carry more scum forming materials than others, and any concentration effect of the clarifier sludge removal equipment probably accentuates the scum forming tendencies of the sludge itself. Scum forming materials that cause trouble in a digester may be classified approximately as follows:

1. Grease and oil products such as:

(a) Waste from meat packing houses, soap factories and kitchens, usually classed as digestible materials.

(b) Wastes of petroleum origin from oil refineries, machine shops, garages and fuel oil tanks, usually classed as indigestible.

2. Mat-forming materials from industrial plants such as:

(a) Fibers from textile mills, hat factories.

(b) Animal hair from tanneries, slaughter houses.

(c) Paunch manure from meat packing houses.

(d) Rinds or skins from fruit packing houses.

3. Miscellaneous indigestible materials such as twigs, milk bottle caps, matches, rags, etc.

4. Gas-buoyed sludge.

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Scum of Industrial Origin

Where the scum is due to industrial wastes, the operator has two courses to follow in solving the problem. One is to fight it out at the plant with the hope of eventually overcoming the difficulty. The other and more effective course is to remove the cause at its source. This might seem like a hopeless task to an operator, but it is surprising to realize how many times it has been done with excellent results.

The smaller the sewage plant, the more likely this method has of success, because in such places the scum producing material usually originates in a single offending industry. At Danbury, Conn., for example, the sewage treatment plant was in perpetual trouble several years ago with a heavy mat of scum in the digester caused by small particles of fur in wastes from local hat factories. The operator had to resort to removal of the manholes and forking this material out of the digester by hand at frequent intervals, and efficient operation was impossible to say the least. Installation of a fine screen by the city and disposal of the screenings elsewhere kept the bulk of this material out of the digester and greatly reduced the problem.

At Sheboygan, Wis., another example may be cited where a local tannery contributed a waste containing a heavy load of hair mixed with lime. While the lime aided sedimentation in the primary treatment plant, by the time the mixture reached the digesters it formed a solid mat of scum and plaster-like sludge. After a few years of this the operator convinced the officials that the plant would not function properly and the result was the installation of a fine screen at the tannery. It solved the scum problem at the sewage plant and enough lime is still present in the sewage to aid sedimentation.

There are many examples of pretreatment of packinghouse wastes by fine screens, as at Cedar Rapids, Iowa, Sioux Falls and Huron, South Dakota, which have kept the major scum forming materials from reaching the city plant. Many canneries have also installed equipment to remove scum forming materials before discharging their waste into the municipal system, and by so doing they have relieved the sewage plant operator of a major job.

These examples result from effective cooperation of industry and city officials and are usually to be found in a smaller community where the industry is a dominant factor, but where it does not require a separate waste treatment plant of its own. Thus, it may be said that the best method of control of scum produced by industrial wastes is to keep the scum forming materials out of the sewage plant.

In larger cities, however, scum producing materials usually find their way into the plant regardless of what the operator or city can do. Even the smaller plants without industrial wastes occasionally have scum producing materials in the sludge, often caused by grease from kitchens. The operators at such plants have a problem which probably cannot be solved by attempting to go to the source. The simplest solution for them is to attack the problem in the plant itself.

Control Measures at the Digester

Scum caused by accumulations of digestible grease and oils can often be eliminated entirely, or conditions greatly improved, by raising the temperature of the digester to 95° or even 100° F. Occasionally, however, when this is attempted in a plain tank, the lower liquid zone heats up but the heat fails to penetrate the scum layer on top. Obviously, the layer must be broken up by some physical means.

One usually effective remedy is to recirculate the digester contents from the bottom to the top, if possible locating the discharge above the scum layer. Incidentally, this operation was covered by process patents now expired.

Where circulation is impossible and no other facilities are available, live steam or hot water injected into the top has been successful. Some operators in despair have even had to resort to breaking up the scum layer with a fork or other tool, working through the digester manhole.

Once the material has been broken up and thoroughly heated, it is then a matter of keeping it from reforming. This is not too difficult because many operators have had no recurrence of troublesome scum formations, once they have learned how to cope with them. Operators at numerous plants report no scum problems because they do not allow it to form. They may keep it broken down by mechanical means or by circulating pumps of adequate capacity.

Scum may also form from accumulations of miscellaneous materials such as twigs, bottle caps, rags, and other things which fail to digest, no matter how long they are held in the digester. They may form a dense layer and, unless removed with the sludge, eventually they will occupy a major portion of the tank volume. This material is not always a real floating scum, as in some plants it appears to be submerged and buoyed by a denser sludge layer underneath. More than one operator has discovered this layer of material after several years of operation, and the only successful way to remove it has been to empty the tank. This has proved a difficult task and one to be avoided by all possible means. To prevent this condition, the operator should sample his tanks at intervals, and determine whether this undigestible material is present and whether it is increasing in volume. The desirable objective is to withdraw this material with the digested sludge, but this is not always possible because its location is not always known.

Gas-buoyed sludge particles occasionally accumulate to form a dense black scum of considerable depth, even where other types of scum may be absent. The best remedy for this situation is agitation, sufficient to separate or scrub these particles, thereby releasing the entrained gas after which the sludge particle should sink. Digested sludge particles in a dry state have a theoretical specific gravity from 1.4 to 1.6 times that of water, so the tendency to settle is present, if the buoyant forces can be detached from the particles.

Digester Design for Scum Control

So far, all suggestions at control of scum have been directed to the operator, but the operator at best can deal only with the installation as it has been turned over to him. He can make minor changes, of course, but usually he must make the best of what he has and cannot count on making major changes until he has worn himself out trying to make the original layout do. Undoubtedly, he could offer some valuable suggestions to the designing engineer, but this is too late so far as his plant is concerned, and anyway the engineer usually has to guess at the type of sewage the plant will actually receive because the mixture of sewage reaching the new plant frequently does not even exist before its completion. From the writer's observations, however, possibly a few suggestions on design will serve in bringing about improvements in facilities provided to the operator for coping with this problem.

Scum Removal Piping

The construction of the digester appears to have some bearing on the scum problem. Several years ago, in the construction of single stage fixed cover digesters, it was not uncommon for the engineer to design as a part of the gas take-off manhole near the center, a scum draw-off connection going directly to the sludge bed. The operator had only to open a valve and any accumulated scum was drained off. These digesters were usually equipped with slow moving scum arms which, with the slope of the roof, brought any floating scum to this manhole at the high point.

Many of these plants operated for years with no scum problem which could not be easily handled. This solution seems so simple yet such a design is now practically extinct.

Scum Breakers

Another design factor is the type of roof or cover used on the digester. Two major classes of roof design are commonly used, one in which the cover is submerged, either fixed or floating, and the other in which the cover is not submerged, and this also may be fixed or floating. An example of the latter type is a gasholder, in which the intervening space is filled with gas under pressure.

To cope with the more difficult problems in the submerged type with fixed roof, the slow-moving type of scum breaking equipment, with alternate fixed and moving fingers passing through the scum zone, has proven very effective. These fingers sweep the entire submerged underside of the roof, thereby preventing formation of thick scum layers and assisting in the uniform release of gas. Installations of this type have been in operation for 15 years without the water level ever having been lowered. This type has also been successful in handling scum problems where garbage and sludge are digested together.

Surface Agitation

Where the cover is not submerged and the liquid surface is free, and where the scum forming materials can move laterally without restraint of the roof surface, some form of surface agitation can be effectively applied. Effective surface agitation, however, cannot be secured without expenditure of power, whether it is done by properly designed mechanical mixing or by recirculation pumps. Within limits, the greater the power applied the greater the effectiveness of agitation. It seems more efficient to apply this power to a mixer located in the scum zone of the digester than to a pump located outside, with attendant losses in valves and piping. Also, a properly designed mixer can agitate the entire scum, which is difficult with any practical pumping arrangement.

This does not necessarily mean greater overall power consumption because experience has shown that intense agitation for short periods is probably more effective than prolonged gentle agitation. For example, in Germany before the war, Prüss installed a 40 h.p. mixer in a digester with an area equivalent to about a 40-ft. diameter tank. The mixer was operated about 30 minutes a day and, according to reports by Dr. Fischer, was very successful in keeping down all traces of scum. This may be going to extremes, but it shows what is possible.

Another illustration of the effect of agitation of a less violent type is at Winnipeg, Manitoba, as described in *Water and Sewage*, February, 1946, in an article by Supt. D. L. McLean. At this plant there are four digesters, of which two were originally equipped with stirring devices and two were not. It was found that, because of scum troubles, the two units without scum equipment handled only half the solids that the other two units with scum equipment did. After several years of operation, scum breaking equipment was installed in the plain units. As a result, the net increase in effective digestion capacity was equal to one complete digester. In this plant the covers are not submerged.

Digester Capacity

Another important engineering factor in design which seems to have a bearing upon scum formation, and on operation as well, is the capacity provided in the digesters. Careful study of a large number of plants leads to the conclusion that those plants with a limited digester capacity of less than 25 to 30 days, based on the volume of the raw sludge feed, almost invariably have trouble with scum or supernatant or both. Those plants with digester capacities of 50 or 60 days or longer almost invariably have no serious scum or supernatant problems. In other words, providing a liberal sludge digestion capacity seems to go a long way toward solving scum problems.

Scum Resulting from Sludge Thickening

There is one source of scum trouble in digesters which appears to be a combined design and operating problem. It may result from lack of capacity or from the zeal of the operator. This refers to the prevailing practice of trying to thicken sludge in the digesters. This is an understandable desire in many plants where digested sludge is to be filtered, or where sludge bed capacity is limited.

Any such thickening of the digested sludge, however, usually results in a deterioration of the quality of the supernatant, and when the supernatant is bad, scum troubles can be expected. As a rule, the shorter the digestion period, the greater the tendency to want to thicken, and the more the sludge is held back in the digester for thickening, the more the supernatant and scum are adversely affected. Part of the remedy for this condition is to provide greater capacity in the digester, and part of it is to thicken the sludge in the clarifier or thickener before it is put into the digester. Then, having done this, if the operator also makes certain that he removes, via the sludge outlet, the digested sludge at the rate at which it is accumulating, one major cause of scum formation, and supernatant troubles as well, will be eliminated.

Summary

Summarizing, scum control in digesters is a broader subject than it might appear to an individual operator faced with the problem in his own plant. Scum prevention by removal of scum forming materials from industrial wastes, preferably at the source, is very effective and lasting in benefits.

The operator, depending upon the facilities available in his plant, can obtain relief by applying more heat, by recirculation, by mixing, and by using scum draw-off connections. The designing engineer can design greater digester capacity into the plant and also provide a design which will allow scum formation once formed to be adequately dealt with.

In any plant the final burden is upon the operator. If he makes certain that everything he puts into the digester as raw sludge is ultimately removed as gas, supernatant or digested sludge, his problems are bound to be fewer.

SCUM CONTROL IN DIGESTERS—A DISCUSSION*

By HARRY E. SCHLENZ

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The paper presented on the subject by Mr. Rankin gives a clear picture of the "scum problem" and includes a complete analysis of the types of scum which may be encountered in digesters.

We are mainly interested in the grouping of scum forming matter into two general classifications, namely: (1) non-digestible accumulations, and (2) digestible accumulations.

Non-digestible accumulations in digesters of a plant receiving a normal domestic sewage represent such a small amount of material that no particular problem is encountered with them. Mr. Rankin has covered quite completely the question of scums from sewages containing a large percentage of non-digestible material due to industrial wastes.

The grouping of scums under "digestible accumulations" represents the major bulk of floating material encountered in the normal digestion installation, and is the type with which plant operators are mainly concerned in connection with "control measures." Such scum accumulations may be generalized into the following characteristics:

(a) High volatile content, ranging from 70 per cent to 90 per cent volatile solids.

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(b) Contain a large percentage of fats and soaps.

(c) May become quite compact and stiff, with as high as 25 per cent solids, if not controlled.

Until recent years, it has been the practice of many plant operators to avoid adding scum or greases, removed from sewage by clarifiers or grease separation tanks, to the digestion system, to attempt to avoid scum accumulations in digesters. Many digesters are equipped with scum draw-off pipes, which are used to remove and dispose of scum accumulations outside of the digestion system.

Actually the digester is the ideal place to dispose of greases and scum, and the emphasis should not be placed on the difficulties encountered with scum accumulations, but rather on the benefits that may be derived from this material as far as gas production and the composition of such resulting gas are concerned.

In any digester where raw solids are added, scum will tend to separate and form at the top of the tank. As long as the scum thus formed does not exceed a foot or two in depth, and is of a soft consistency, there is good indication that it is not accumulating and is undergoing normal digestion. In digestion systems receiving an abnormally high loading of raw solids, or where the grease content of the sewage is high, scum may build up quite rapidly, with such scum accumulations seriously affecting the digestion of the solids in the balance of the tank, by reducing the volume available for active digestion.

Control Procedures

Although as recently as ten years ago the literature gave evidence of the doubt of investigators that greases could be digested within any reasonable length of time, it is now definitely indicated that greases are readily digestible, with resultant higher percentage reduction of the volatile content of the greases than of the balance of the volatile solids in the digestion system. It is therefore evident that if the proper conditions are provided in the scum mass, it is possible to promote the rapid destruction of the fats and soaps present. In the destruction of these materials by digestion, lower fatty acids are first formed, which digest further to produce methane and carbon dioxide.

The extent to which control measures must be employed to promote the destruction of scum accumulations is dependent upon the amount and character of the scum forming matter in the sludge added to the digestion system, and may involve one or more of following procedures.

Liquid Saturation

Scum accumulations should be kept in intimate contact with the liquid contents of the digester, which contain active organisms to promote the desired rapid digestion. This may be accomplished by:

(a) Positive submergence, by an impervious structure of the materials which tend to float, thus preventing the scum from rising into the gas zone where it may become permeated with this end product of digestion and tend to become dry to some extent. Such submergence, which will suffice in preventing the building up of scum accumulations in the majority of digester installations, does not require the expenditure of power, and even where additional control measures are required, the advantages gained through submergence would indicate the feasibility of its use.

(b) Recirculation of digester liquor from portions of the tank contents where solids are undergoing the most active digestion, with the discharge of the recirculated material into the scum zone, as shown in Figure 1.

In practicing such liquor recirculation, even when confronted with very troublesome scum conditions, it has been demonstrated that it is not necessary nor desirable to discharge the recirculated liquor into the scum with any appreciable force. The rate of discharge should be such as to cause the recirculation of the volume of the upper two-thirds of the digester in from 24 to 48 hours, either by continuous or intermittent operation of a non-clog pump requiring a low power input. The most completely digested sludge in the lower one-third of the tank should not be disturbed, so as to be available for transfer to a secondary digester or removal to a point of final disposal.

While the writer was engaged in the operation of the sewage testing station at the University of Illinois in 1929, a digester of pilot plant size, when operated at a comparatively high loading of raw solids, persisted in the formation of large quantities of scum. Efforts to control such scum accumulations by slow moving paddles or rapidly moving mixers did not

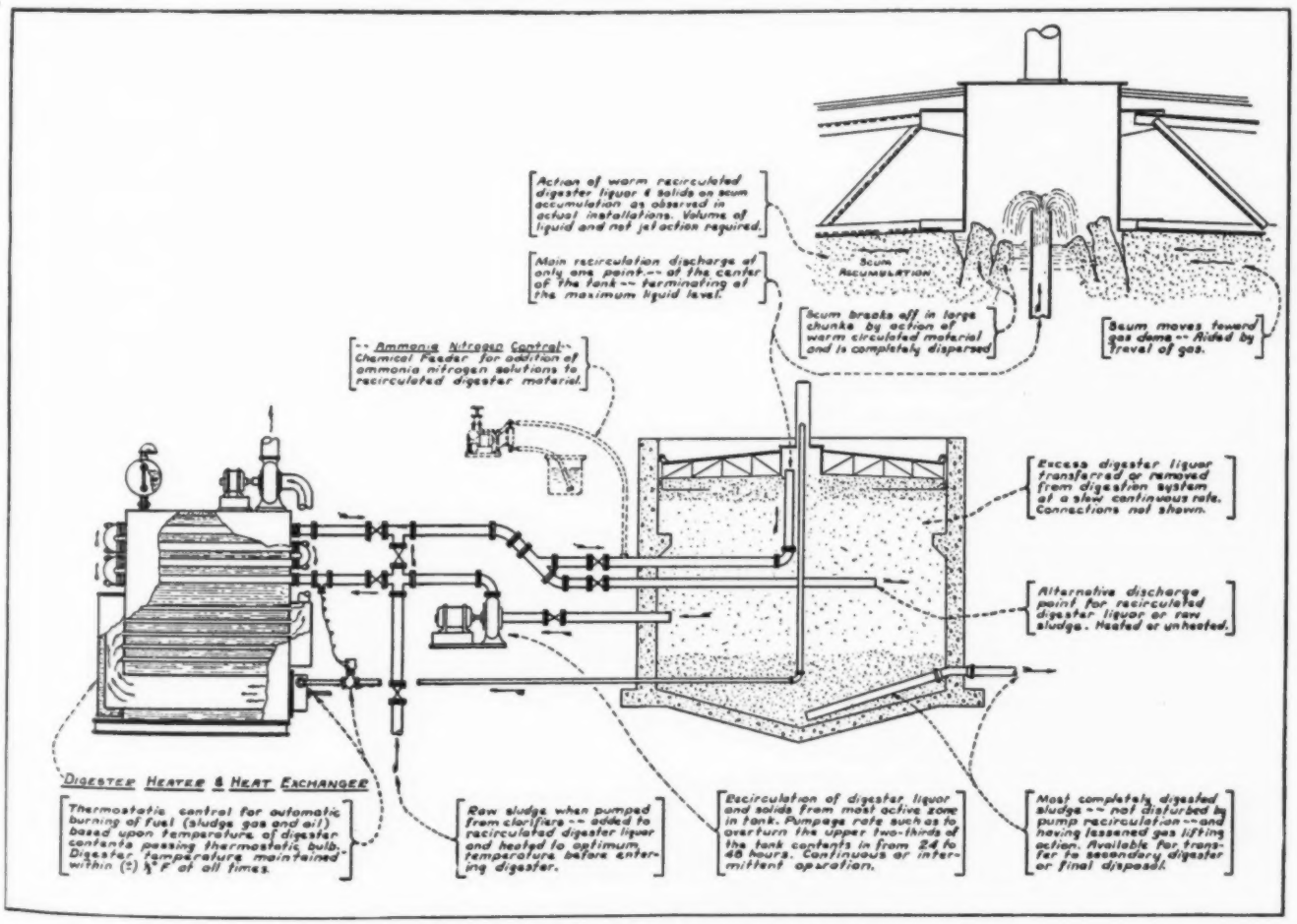

Action of warm recirculated digester liquor & solids on scum accumulation as observed in actual installations. Volume of liquid and not jet action required.
Main recirculation discharge at only one point -- at the center of the tank -- terminating at the maximum liquid level.
Scum
Accumulation
Scum breaks off in large chunks by action of warm circulated material and is completely dispersed
Scum moves toward gas dome -- Aided by travel of gas.
-- Ammonia Nitrogen Control -- Chemical Feeder for addition of ammonia nitrogen solutions to recirculated digester material.
Excess digester liquor transferred or removed from digestion system at a slow continuous rate. Connections not shown.
Alternative discharge point for recirculated digester liquor or raw sludge. Heated or unheated.
Digester Heater & Heat Exchanger
Thermostatic control for automatic burning of fuel (sludge gas and oil) based upon temperature of digester contents passing thermostatic bulb. Digester temperature maintained within (±) ½° F at all times.
Raw sludge when pumped from clarifiers -- added to recirculated digester liquor and heated to optimum temperature before entering digester.
Recirculation of digester liquor and solids from most active zone in tank. Pumpage rate such as to overturn the upper two-thirds of the tank contents in from 24 to 48 hours. Continuous or intermittent operation.
Most completely digested sludge -- not disturbed by pump recirculation -- and having lessened gas lifting action. Available for transfer to secondary digester or final disposal.

Fig. 1.—Diagram Illustrating Methods of Scum Control in Digestion Tanks.

meet with any great amount of success. It was found, however, that the stiff scum could be dispersed by a small stream of recirculated liquor with an expenditure of power of but a fraction of that used in the mixing procedures. It was further found to be more effective to discharge the recirculated liquor at the center of the gas take-off dome at the top of the scum layer, rather than at a number of points in the scum zone.

During the war period many digester installations at camps were confronted with scum problems in concentrated form due to the high grease content of the sewages encountered.

Observations made over an extended period of time in connection with scum control in the digesters at one of the two plants of the Great Lakes Naval Training Center were reported in an article on "Controlled Digestion" which appeared in the May, 1944, issue of the *Sew. Wks. Journal*. Due to improper starting of digestion and before scum control measures were started, each of the two 65-ft. diameter digesters contained about 9 ft. of yellow sickly-looking scum which had become so concentrated that it was impossible to penetrate the mass with a two-by-four. Digester liquor circulated at a rate of 300 g.p.m. caused the following action to take place: The area of the scum mass immediately adjacent to the relatively gentle discharge of the recirculated liquor first became soft and mushy; then large masses of scum resembling a firm fresh cheese gradually moved from the underside of the floating cover towards the gas dome and broke off in large chunks as they became softened. This action continued and after a few days it was evident that the entire scum mass was of a soft mushy consistency, completely permeated with the digester liquor.

In an article appearing in the September, 1946, issue of *Water and Sewage Works*, page 369, R. W. Simpson, formerly Sanitary Engineer at Keesler Field, Miss., reports that digester recirculation was effective in dispersing scum accumulations which were an ever present problem, even though the primary digester was equipped with mechanical mixing equipment. He states, "as long as the scum was kept soft, digestion and settling took place. It was necessary to use this (recirculation) method several times, but in each instance the scum thinned out and completely disappeared shortly thereafter."

Control of Temperature of Scum Zone

The writer agrees with the statement that temperatures of 95° F. or more are effective in softening the greases in scum accumulations. However, it is important that this temperature be maintained in the scum forming zone, as well as in the other portions of the tank. A study of the "inventory" records of digesters characterized by excessive scum accumulations reveals that the temperatures in the scum zone may be 10° to 15° lower than the temperatures which are maintained in the balance of the digesting mass.

Liquor recirculation as previously described, in addition to bringing seeding or actively digested solids into the scum accumulations, also tends to provide the desirable optimum temperatures for digestion in the scum zone.

It is possible by the provision of external heating means to raise the temperature of the recirculated digester contents, and thus greatly increase the rate of digestion of the scum by the discharge of the higher temperature liquor at the surface of the scum layer in a manner as shown in Figure 1. With a heating system of this nature provided with sensitive thermostatic controls it is possible to maintain automatically the temperature of the entire digester contents, including the scum zone, within ±0.5° F., and thus provide optimum conditions favoring the growth of desirable methane producing organisms.

Furthermore, it is desirable to lessen or eliminate entirely the tendency of scum-forming materials to separate from the raw solids as they are added to the digestion system. This may be accomplished by adding the raw solids to the recirculated digester contents, and then heating the mixture so that on entering the digester initially, the sludge is properly seeded and heated to the optimum temperature for digestion and completely dispersed, thereby accelerating the rate of digestion.

Ammonia Nitrogen Control

In a number of digester installations where especially heavy scum accumulations have been encountered, the addition of relatively small amounts of an ammonia nitrogen compound to the liquor recirculated to the scum zone has been effective in causing the scum to digest quite rapidly and to disappear.

Ammonium sulfate has been used as one source of ammonia nitrogen, the commercial product yielding about 25.6 per cent of ammonia nitrogen. This ammonia salt is readily soluble in an equal weight of water and is easily obtained at low cost.

It is added in relatively small doses of 50 to 100 lb. with continuous liquor recirculation between doses. Additions are made until about 30 p.p.m. of equivalent ammonia nitrogen has been added to the volume of the tank being recirculated, normally assumed to represent one-half of the tank contents.

It has been found that if the ammonium sulfate is added at too rapid a rate there is a tendency for the activity in the scum zone to reach such proportions as to resemble a "foaming" condition. The continued addition of the ammonia salt to an excessive concentration of ammonia nitrogen in the digester may also be detrimental to the digestion of the scum solids.

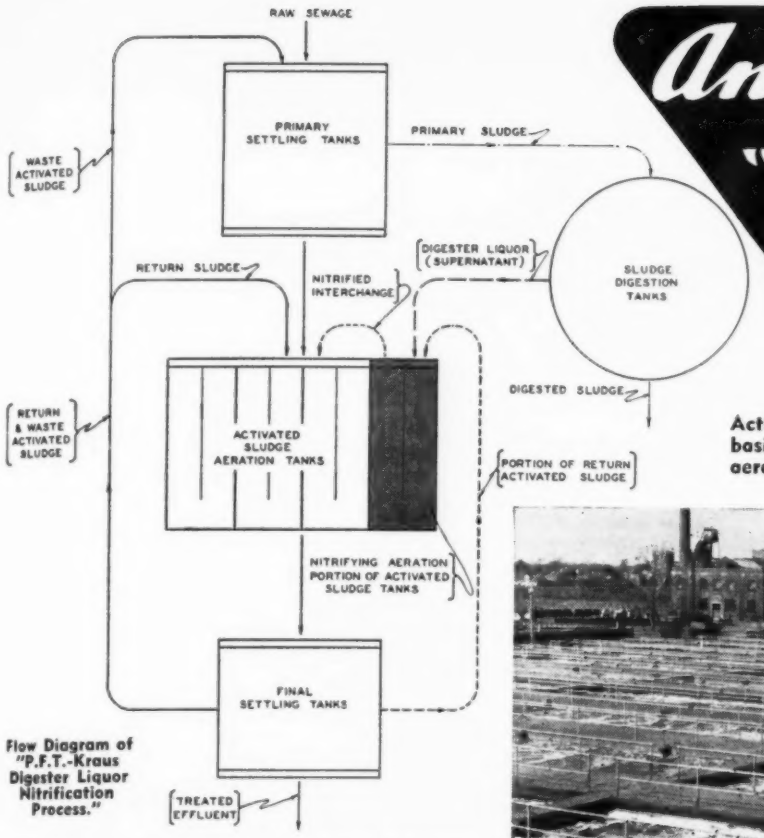
At the large naval installation previously referred to, the raw sewage contained abnormally large quantities of grease, which, when added to the digester, resulted in such a large bulk of scum that digester recirculation alone was not sufficient to keep it dispersed at all times. In the last two periods of operation in the tabulated data as reported in the article on "Controlled Digestion" previously referred to, it was indicated that 10 days of continuous recirculation of liquor into the scum zone slowed down the rate at which scum tended to accumulate, but within 6 days after the addition of only one 100-lb. dose of ammonium sulfate, representing 7 ppm. of ammonia nitrogen added to the tank contents, the scum accumulation was completely dispersed. During the past two years the plant superintendent at that installation has confirmed these observations many times, and reports that he feels that he has a positive method of controlling scum which previously had presented an operating problem of great magnitude. His procedure is to practice more or less continuous liquor recirculation, which keeps scum accumulations at a minimum and the digestion activity at an optimum point as measured by rate of gas production. Whenever the scum accumulations become excessive in his judgment, he adds 100 lb. of ammonia sulfate to the recirculated liquor and the scum is completely dispersed in a matter of a few days. He has found that on the average it is only necessary to resort to the use of this small dose about once each three months, representing a cost of less than $2.00 per dose for the chemical.

The writer will agree that it is difficult to understand why the addition of such a small amount of ammonia nitrogen to a digesting sludge mixture that is normally high in ammonia nitrogen content will show such definite beneficial results, as measured by scum dispersal and increased gas production.

It is possible that the scum zone may be deficient in ammonia nitrogen and under some conditions the liquor recirculated from other portions of the tank will correct this deficiency and cause the scum to digest. In cases where liquor recirculation alone does not result in sufficient increased activity in the scum zone to destroy the scum accumulation, the addition of the small quantities of ammonia sulfate to the recirculated liquor probably provides ammonia nitrogen in a readily usable form for the organisms, resulting in the increased digestion rate in the scum zone.

Patent Considerations

Some of the procedures suggested in this discussion, such as digester liquor recirculation and ammonia nitrogen control, are covered by patents already issued, while certain phases of external heating of circulated digester liquor and raw solids are covered by pending patent applications. It is therefore necessary from a patent standpoint for an operator wishing to use such procedures to obtain rights to do so. In addition, ammonia nitrogen control should be used only under the guidance of engineers of the Pacific Flush Tank Company to avoid difficulties which may result from improper use of the procedure.



Flow Diagram of "P.F.T.-Kraus Digester Liquor Nitrification Process."

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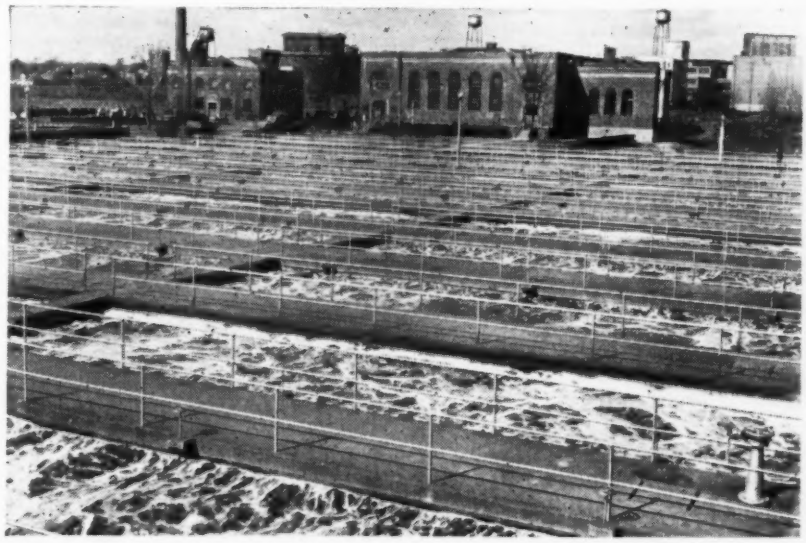

Activated sludge aeration tanks, Peoria, Ill. Three basins in the foreground, out of a total of twelve aeration tanks, used for sludge nitrification.

A MAJOR ADVANCE IN THE CONTROL OF ACTIVATED SLUDGE PROCESSES

Now available and quickly applied to new and existing plants . . . the "P.F.T.-Kraus Process" for positive control of the sludge index in activated sludge plants. Originally developed by L. S. Kraus, Chemist of the Greater Peoria, Illinois, Sanitary and Sewage Disposal District, the process has thoroughly proved itself in successful operation for several years. In the Peoria plant, handling domestic sewage from a population of 115,000 together with industrial wastes of equal or greater population equivalent, the process has:

(1) Eliminated the need for expanding plant facilities by overcoming the effect of greatly increased loads exceeding the original plant capacity.
(2) Nullified the formerly disastrous effect of shock loads from industrial waste organic matter.
(3) Utilized all digester liquor overflow to advantage in the main treatment process.

Also known as the "Digester Liquor Nitrification Process," it does not require additional aeration basins, aeration facilities or any greater amount of air. It merely calls for a slight revision of piping and return activated sludge pumping facilities. Using this system, the sludge produced in the activated sludge process settles more readily to give better overall results in purification or allows reduction in the capacity of secondary settling facilities. The process effectively overcomes another problem by making use of digester liquor to advantage.

Digester liquor or digested sludge, or a mixture of the two, is added to the activated sludge process in a portion of the aeration tanks set apart as nitrifying aeration tanks as shown in the above diagram. The ammonia nitrogen in the digester liquor or sludge is converted to nitrites and nitrates. This produces, in the nitrifying aeration tanks, a sludge containing a high concentration of highly nitrified materials which are then returned to the remaining activated sludge aeration tanks. The highly nitrified sludge may be returned continuously to the normal activated sludge process or may be returned in amounts required to balance shock loads of organic matter introduced into the aeration tanks.

Other plants installing the process include: Northeast Sewage Treatment Plant, Philadelphia, Penn.; South Bend, Ind.; and Charlotte, N. C. Pacific Flush-Tank has the exclusive rights for licensing this process and is prepared to supply immediate recommendations and data.

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SEWAGE TREATMENT EQUIPMENT EXCLUSIVELY SINCE 1893

SEWAGE SLUDGE AS A FERTILIZER*†

By A. H. NILES
Superintendent, Division of Sewage Disposal, Toledo, Ohio

NEW operators of sewage plants have often been amazed at the lush growth of garden produce which springs up as volunteer plants around sewage works, many times growing in the partially dried sludge cake undiluted with soil. This observation probably occurred most frequently with tomato plants, which as they grew were much sturdier, much deeper green and much larger than those which happened to grow in the surrounding soil unaffected by sewage sludge.

This new operator probably had only a passing interest in the appearance of these plants but as the season progressed and the fruit began to ripen, the day came when he was perhaps short of sandwiches or was long on appetite. The appearance of a particularly large and luscious looking tomato became irresistible. He had become sufficiently well acclimated to the atmosphere of a sewage treatment works to overcome his natural aversion to the thought of eating a fruit or vegetable which was fertilized directly by human feces, whether fully digested or not.

As this new operator sampled the appealing fruit, he smacked his lips and probably murmured "Hmm—not bad." From that time on he was sold, and being more or less a loquacious individual, did not wish to hide his discovery "under a bushel." He told his friends and they in turn told theirs so the story of sludge as a fertilizer began to be a matter of not uncommon knowledge. However, as in most cases of this kind, there was considerable misinformation disseminated along with the truth. Soon extravagant claims were made that sewage sludge was the universal panacea for all the ills of the agricultural world. Unscrupulous promoters claimed that sewage plants could be made self-supporting solely by the sale of their sludge as fertilizer.

Farmers long had believed that the only difference between the commercial fertilizer manufacturer and Jesse James was that the commercial product gave no visible evidence of the company having horses, and the salesmen usually did not wear spurs. Soon the commercial fertilizer companies were worried to the point where they were putting out their own brand of propaganda. These statements were that sewage sludge was of such low value that it was not worth hauling home or spreading on the fields even if given to them. Also, that it was full of disease producing bacteria and the user of the produce would certainly contract horrible afflictions or death. As is almost universally the case, the truth, probably, lay somewhere between these two widely divergent viewpoints.

Early Work at Toledo

The writer, being of Welsh extraction, which according to some, is a Scotchman raised to the *n*th power, felt that it was economically wrong to throw away anything of demonstrated value. When the Toledo sewage treatment plant was started, it was decided to give the sludge away for a limited time for advertising purposes. This was all right except for the fact that a few persons obtained sludge which was insufficiently dried. When this was spread upon their lawns and their small children ran across the yard and tracked the sticky mess into the house, their irate mothers promptly consigned the sludge and all persons connected therewith to the place where sludge drying could be most economically accomplished. These unfortunate incidents, together with the psychological fact that, to many persons, anything obtained "for free" has no value, gave the sewage sludge program something of a setback before it really got under way.

A start was made, however, with an attrition mill borrowed from a miller friend. It was not well suited for the purpose but did a passably good job. This was followed by a threshing machine concave and that in turn by a light farm hammer mill primarily suited for grinding grain and feed. Quite a lot of tonnage was put through that old farm mill and the product packaged in used calcium chloride burlap bags obtained from the Street Division. This package was an unattractive one and, besides, its printing indicated that it contained a product to lay dust or melt ice instead of to grow plants. It was sold to the relatively few persons who had previously used the free sludge and were sufficiently pleased with the results to part with real money. This was especially noteworthy because it occurred at a time when pocketbooks were flat—when the CWA leaf-raking program was at its height and national budgets were ending with six zeros instead of nine.

Finally, in 1937, a WPA project was approved to construct a plant to prepare economically the dried sludge for fertilizer and a warehouse to store it in. A contest was initiated in June, 1938, to name the child formally. Many names submitted were not thought usable because of being too expressive, but as a final result, the infant was christened "Tol-e-Gro." Its godfather was rewarded by a handsome prize of 10 bags of the material—a part of which he promptly gave away to his close friends and neighbors.

A multiwall paper bag, printed in two colors, with analyses and complete instructions for use was adopted together with modern equipment for preparing, bagging and handling. A modest newspaper advertising program was inaugurated and a salesman hired for two seasons to get the merchandising campaign under way. Most of this took place in 1938 and 1939. Since that time the only advertising done has been the printing of folders used by dealers to acquaint their customers with Tol-e-Gro. No salesman has been needed since 1939, for every bit of Tol-e-Gro that could be dried has been sold. In the spring of 1943 at least 250 tons more could have been sold if it had been available.

Just about this time, in the Fall of 1937, the Toledo plant had the good fortune to obtain a chemist who had the background, the training, the energy and the inquisitive mind to do a good job of research. His assignment was to find out what made sewage sludge "tick." It is the purpose of this paper to tell you some of the things he found out.

Fertility Factors in Sludge

First of all, from general observation, it was assumed that sewage sludge had something that promoted growth other than the amounts of nitrogen, phosphoric acid and potash that could be determined from analyses in the chemical laboratory. All that is necessary to confirm this observation is to read the history of China. The Chinese have used their waste products—human, animal and vegetable—for centuries as fertilizers. Their lands are still among the most fertile in the world. Chemical fertilizers were unknown to these uninformed people, yet they maintained the fertility of their land. Their country was too crowded to move to more fertile, virgin fields. It was absolutely essential that they maintain their soil fertility. To do so they used the above mentioned wastes, putting them back into the soil. In Europe, the farmers of today will not tolerate the waste of manures—even human manure is carefully used for fertilizer. These farmers, undoubtedly, knew very little and cared less about how much nitrogen, phosphoric acid and potash were in their wastes. However, they did know that these wastes were what their ground needed to keep it fertile and productive. These manures and natural wastes actually were poor fertilizers if compared analytically with some of the so-called complete fertilizers, such as 4-12-4 or 5-10-10, etc.

It is true that these chemical fertilizers are giving good results. But is it not reasonable to believe that these natural wastes must have something in them or their action which makes them better fertilizers than the straight analysis of nitrogen, phosphoric acid and potash would indicate? No one can make up a standard mix commercial fertilizer with the same analysis as Tol-e-Gro, which is 2.0-2.0-0.30, and come anywhere near to obtaining the results that can be obtained by using Tol-e-Gro or a barnyard manure with a similar analysis. Immediately, it will be said that the organic form of nitrogen is of longer duration and the plant fed more continuously, but that again is not the whole story. The purely chemical analyses in the laboratory do not show the countless billions of little nitrogen fixation factories found in sewage sludge and manures which take nitrogen from the air and convert it to the form of nitrogenous compounds easily assimilable by the plant life. Tol-e-Gro contains enormous numbers of nitrogen fixation bacteria which are found in some soils in limited quantities but not in all soils. Their presence is conducive to good fertility. These bacteria are collectively known as Azotobacters and Nitrobacters. These micro-organisms have the power of transforming the nitrogen in the air to useful compounds which can be assimilated by plants for food. They not only make nitrogen more available to plant life but they also attack and break down humus, straw, old dead roots and other organic substances in the soil, to useful, life-giving stimulating compounds used by the plant life.

*Reprinted from *Sew. Wks. Jour.* by permission.
†Ed. Note: Although this article was prepared about and mentions Tol-e-Gro, the sludge from Toledo, Ohio, the information herein is undoubtedly applicable to digested sludge from other sewage plants.

Nitrogen and Nitrate Content

It was found that Tol-e-Gro will increase its nitrogen content by 20 per cent in 7 days if moistened and allowed to stand exposed to air. This nitrogen in the air is changed to ammonia and nitrates which are eagerly taken up by the plants and utilized in their normal metabolism. The Texas Experimental Station Bulletin No. 445 states that similar sewage sludge mixed with samples of different kinds of soils produced from 64 to 134 parts per million of nitrate nitrogen. This is more nitrate nitrogen than is produced in some higher analysis fertilizers. This variation is due largely to the variations of soils rather than to the sewage sludge itself.

Since a medium or normal nitrate soil will show 5 to 15 parts per million, according to Michigan Technical Bulletin No. 132, it is obvious that sewage sludge gives an abundant supply of nitrate nitrogen. Quoting from the above Bulletin No. 132, "The nitrogen of the soil organic matter is transformed to ammonia and then to nitrites and finally to nitrates by means of the action of soil organisms. This process is known as nitrification. In order for a soil to form nitrates naturally, it should have a supply of nitrogen in organic matter, be moist, warm and aerated and have the nitrifying organisms present. Soils low in organic matter are usually low in nitrate and nitrifying organisms. Growing plants obtain most of their nitrogen from the soil as nitrates." Nitrates are water soluble and, unless protected by organic matter, are easily washed through the soil.

Tol-e-Gro supplies an abundance of these nitrifying organisms as well as a good reserve supply of organic matter and prevents leaching of valuable nitrate salts from the soil. The amount of ammonia in the soil is dependent upon the same factors as nitrates, that is, abundance of organic matter and nitrifying organisms. Chemical salts used on soils are easily leached out by heavy rains and partly wasted as far as soil nutrition is concerned.

The use of sewage sludge or manure will, in a large measure, control this leaching as it will hold many times its volume and weight of moisture, thus keeping the dissolved nutrients in contact with roots for a much longer time than would otherwise be possible.

Most sewage sludges have a residual or reserve property of releasing food as the plant needs it. These organisms will make just a certain amount of excess ammonia or nitrate which is more than the plant needs. They cease production of these nutrients and cease their activity until the nitrate concentration falls somewhat. Then the nitrifying bacteria start making more ammonia and nitrate from the air. Thus, there is always available an ample supply of usable nitrogen.

Sewage sludges and manures are helpful in making heavy clay soils porous and workable. Sandy soils are helped by the humus present in sewage sludge and by the moisture-holding properties of the sludge. Sandy soils are almost always deficient in nitrifying bacteria and certain colloidal conditions which are essential to plant growth. Tol-e-Gro supplies the soil with microorganisms and helps correct these colloidal deficiencies.

Chemical Constituents

Many elements are essential to plant growth other than nitrogen, phosphorus and potassium. Many crops have been thoroughly analyzed to find just what elements are needed in their growth. Analyses were made of corn, wheat, sugar beets, alfalfa, tomatoes, barley, rye and cabbage, as reported in *New Agriculture*, April, 1937. The following elements were found to be present in the above crops: nitrogen, phosphorus, potassium, calcium, aluminum, arsenic, barium, beryllium, boron, bromine, chlorine, chromium, cobalt, copper, fluorine, iodine, iron, lead, lithium, magnesium, manganese, molybdenum, nickel, rubidium, silicon, silver, sodium, strontium, sulfur, tin, titanium, vanadium and zinc. This totals 33 elements in all. Of these elements, 22 are present in corn, 26 in wheat, 20 in sugar beets, 27 in alfalfa, 22 in tomatoes, 14 in barley, 15 in rye and 22 in cabbage. Therefore it seems reasonable to assume that a good fertilizer should include more than just nitrogen, phosphate and potash. Tol-e-Gro shows the presence of 29 of these various elements.

Sewage sludge has an abundance of mineral elements, many of which are present in such forms as to be slowly and sufficiently available to plant life. Since about 30 different minerals are being constantly taken from the soil by crops, it seems reasonable that replacement of many of these elements is advisable.

A composite sample of 325 tons of Tol-e-Gro was carefully collected and analyses made of same sample. These analyses are quite complete for all common mineral and non-metallic constituents, and the results, shown in the accompanying table, are expressed in pounds per ton of Tol-e-Gro:

Chemical Constituents of Tol-e-Gro.

Constituent	Pounds per ton	Constituent	Pounds per ton
Aluminum	72.0	Lithium	Tr.
Arsenic	.2	Magnesium	22.0
Barium	1.0	Magnesia equivalent	36.5
Bismuth	12.0	Molybdenum	Tr.**
Boron	+*	Nickel	Tr.**
Bromine	Tr.**	Nitrogen	42.4
Calcium	94.8	Organic Matter	888.0
Ca as limestone	236.8	Potassium	7.0
Chlorine	+*	Phosphoric Acid	40.3
Chromium	5.0	Selenium	Tr.**
Cobalt	Tr.**	Silicon	146.0
Copper	16.0	Sodium	+*
Fluorine	+*	Sulfur	65.0
Iodine	Tr.**	Tin	Tr.**
Iron	76.0	Zinc	20.0
Lead	15.0		

*Undetermined amounts, but more than a trace.
**Spectrographic analysis confirmed these trace constituents.

It is fitting at this time to take up some of these various elements, showing in some measure just how and why they are vitally important in the metabolism and growth of plants.

Aluminum: Essential to plant growth. It is usually present in soils in sufficient quantity although it may be deficient in sandy soils. Along with the organic matter in the soil, aluminum is active in controlling the colloidal functions and water-conserving properties of the soil.

Arsenic: The following statement is taken from *New Agriculture*, April, 1937, issue: "Although formerly believed to be toxic in certain concentrations, it has proven beneficial and stimulates the growth when present in small quantities. As much as 1,000 lbs. per acre have not proven toxic in experimental work." Dr. J. E. Groves of Utah says "Arsenic stimulates bacterial action in the soil resulting in greater crop yield." He further states: "Arsenic must be applied in enormous amounts before it retards microscopic life." Experiments show that arsenic causes liberation of insoluble elements in the soil, especially phosphate.

Barium: Found in varying amounts in some plants. It is believed to prevent injury from excessive concentrations of other elements. However, there is not much known about the value of barium except that it is found in the structure of some plants, namely, corn, wheat, sugar beets, alfalfa, tomatoes and cabbage.

Boron: Much work has been done on the influence of boron on plant growth in the last 20 years. Warington (1923) and Breachley (1927), at the Rothamsted Experiment Station, believe that boron is essential to plant growth, and this belief is concurred in by Sommer and Lipman (1926), Johnson and Dove (1928) and others in this country. Collins (1927), working at the New Jersey Experimental Station, found that boron did have a stimulating effect when present in small quantities.

Calcium: One of the very essential elements. Non-legume crops will require at least 25 lbs. per acre, while legumes, alfalfa, clover, etc., will require 100 lbs. and more per acre. Most soils, it is true, have calcium in their make-up but it is not always in an easily available form. Calcium, more or less, controls the acid and alkaline balance in the soil. It has a controlling influence on the translocation of sugars and starches in the plant. It is essential to the construction of healthy cell wall of plant tissue.

Chlorine: Apparently useful to plants in small quantities. It seems to have a catalytic effect and tends to make other elements more available.

Copper: Found in many plants and it appears to be essential for sunflowers and tomatoes. Copper gives good results, especially on wet, boggy lands where most minerals have been leached out of the soil. Copper, along with manganese, has decidedly beneficial results with carrots, radishes, beets, cabbage and other garden crops.

Cobalt: In small quantities has been found to stimulate crops. It has been found to be important in tomato, wheat and corn culture (*New Agriculture*, April, 1937).

Magnesium: As essential to plants as phosphorus or potash, yet until recently experimental stations and fertilizer industries have paid little attention to the possibilities of a magnesium hunger or magnesium fertilization. Most plants will show from 5 per cent to 10 per cent magnesium oxide in their ash. Magnesium is a constituent of the chlorophyll molecule. It is absolutely necessary in the production of a healthy green color in plants. Magnesium plays an important role in the translocation of starches in the plant. It is an essential factor in the formation of vegetable fats and oils and functions as a carrier of phosphorus in plant metabolism. Bartholomew (1933) has gained evidence showing that soluble magnesium in plants increases the absorption of phosphorus. Cooper (1930) found that magnesium is desirable for resinous plants. Sewage sludge carries large amounts of easily available magnesium. Loew (1901) showed that for the best development of most common field crops the soil should have a calcium-magnesium ratio of 2 parts of calcium

to 1 of magnesium. During wet seasons magnesium is easily leached from the soil. The combination of magnesium with organic material is, therefore, highly desirable as the organic material helps prevent excessive leaching.

Iron: Very essential but most soils contain sufficient amounts for normal growth.

Manganese: A constituent of nearly all field and garden crops. It is now considered to be one of the essential elements for normal growth. McHargue (1926) of the Kentucky Experimental Station demonstrated conclusively that manganese was essential for plant growth. Manganese appears to be necessary in the normal oxidation reactions taking place in plant metabolism. Addition or incorporation of manganese in fertilizers to be used on alkaline soils or soils underlain with lime is important as the alkalinity reduces the availability of manganese. Therefore alkaline soils are apt to be deficient in available manganese. Manganese deficiency is due, largely, to the increasing scarcity of manure. Manure usually contains sufficient manganese for most soils. Sewage sludge contains an abundange of manganese, even more than manures, and is a very good carrier of manganese for fertilizer use.

Sodium: Easily leached from soils and should be replaced. Beets, mangoes, potatoes, wheat, barley and asparagus respond quickly to the presence of sodium salts in the soils. Some experimental stations are using top dressings of sodium chloride, or common salt, with remarkable increases in crops. Sodium seems to help conserve the phosphorus and potash and help the normal reactions in the plant.

Sulfur: Proved to be an essential element to plant growth in 1860. Sulfur is a constituent of many plant proteins, cereals, etc. Sulfur is a constituent of plant flavors and aromatic oils and is a constituent of mustard oil, onions, garlic and many other flavors more pleasant. Many plants require more sulfur than phosphorus (*Commercial Fertilizers*, Collins, 1934).

The annual loss of sulfur through drainage is estimated at between 20 and 50 lbs. per acre. Many plants will respond remarkably to sulfur. Alfalfa will respond to sulfur even better than nitrogenous fertilizers. Cases of alfalfa increases with sulfur-carrying fertilizers have been reported from 50 to 500 per cent. Sulfur increases root growth in most plants (Crocker, 1933). Garden crops such as cabbage, lettuce, radishes, beans, peas, etc., respond readily to sulfur.

Tol-e-Gro contains sulfur bacteria which change insoluble forms of sulfur to soluble sulfates which are available to plants. Tol-e-Gro is rich in sulfur—65 lbs. per ton.

Small amounts of zinc seem to be important to plant life as zinc is found in almost all of the common crop plants and fruits. Zinc controls mottle leaf and rosette in fruit trees.

Other Benefits

Users and experimenters of sewage sludge as fertilizer have repeatedly called attention to the fact that results obtained by using Tol-e-Gro and some similar products are far better than the nitrogen, phosphoric acid and potash analysis would indicate. The importance of other mineral constituents has been shown in a small way. Taking all these minerals into consideration, the results obtained by using Tol-e-Gro are still beyond those expected from the straight, routine, standard analysis.

This may be partially explained on the basis of increased moisture-holding capacity; the change in soil structure, allowing more air to enter and subsequently stimulating oxidation; introduction of large numbers of bacteria to aid decomposition and better balancing of the nitrification processes and flora of the soil.

It has been found in running laboratory comparative tests that the plants grown with Tol-e-Gro had two or three times the root growth of plants grown under identical conditions with commercial 4-12-4 fertilizers. Also, comparative tests with potatoes grown on good loam soil resulted in a 200 per cent increase for hills fertilized with Tol-e-Gro at the rate of 400 lbs. per acre. The root growth was noticeably greater and the potatoes were larger. A search was started to determine in some measure the cause of these fine results and increased root growth. In recent years considerable attention has been focused on the fact that certain organic compounds or chemical substances are able to increase root growth and stem elongation and to influence changes in plant growth. It was in this direction that the search was started.

It is well known that in the presence of nitrifying bacteria, such as Azotobacter Chroococcum, something was formed which induced increased root growth. As previously stated, Tol-e-Gro is virtually alive with nitrifying bacteria. It has been shown that these growth promoting substances are probably not made by the bacteria but are produced as bacteria break down the organic materials upon which they feed. Tol-e-Gro is rich in these organic substances. This explained, in part, the origin of these chemical substances, but the next thing was to demonstrate their presence and approximate the amounts present. These substances were found to be present in small quantities when expressed as pounds per ton, but this is more than sufficient when compared to the requirements for plant stimulation. These substances were identified as derivatives of indole, a complex organic chemical.

Growth Promoting Substances

Dr. Willem Rudolfs, head of research of the New Jersey Agricultural Experiment Station, Dept. of Water and Sewage, together with Messrs. Heineman and Ingols, reported in *Sewage Works Journal*, Vols. 10 and 12, on various growth-promoting substances found in sewage sludge. Their conclusions were that most growth-promoting substances found in sewage sludge were from indole and skatole in the form of indole acetic, indole propionic and indole butyric acids. Tryptophane is also formed by digesting or decomposing sludge. Carotene and ascorbic acid are present in limited quantities.

Many of these substances are soluble and tend to wash or drain out of the digesting sludge. It was found that concentrated filtrate collected from the underdrainage system of the glass-covered sludge drying beds at Toledo was far more potent in growth-promoting substances than the dried sludge. This corroborated the statement heard many times that "the best part of sludge is drained away."

Damoose at Battle Creek, as did Backmeyer at Marion, Ind., found this to be the case when they advocated using, and actually sold on an applied basis, quite large amounts of liquid digested sludge. This method of use certainly has its limitations, but, where possible to handle it economically, it produces wonderful results. For years Hommon, at Canton, O., has disposed of liquid digested sludge directly to a city-owned farm adjacent to the sewage treatment plant, with phenomenal crop yields.

In conclusion, it may be summed up that, measured by purely chemical analyses, sewage sludges do not compare very favorably with many commercial fertilizers. Sewage sludge probably never will drive commercial fertilizer companies out of business. Many of these companies do use sewage sludge as a base and fortify it with additional chemicals to gain the analysis desired. This, in part, gains for the commercial fertilizers some of the advantages sewage sludges have.

Even the worst critics of sludge, however, unwillingly perhaps, admit that it has some properties that give plants an "oomph" that a standard commercial fertilizer just does not have.

CONTENTS OF HORIZONTAL TANKS AND PIPE

Per Foot of Length, for Various Depths

Formulas for determination of approximate capacity of horizontal cylindrical tanks for any depth. Given: Diameter of tank d and height of segment h.

To find area of segment.

when $h = 0$ to $\frac{1}{2}d$; area $= h\sqrt{1.766\,dh - h^2}$

when $h = \frac{1}{2}d$ to d; area $= h\sqrt{0.017d^2 + 1.7\,dh - h^2}$

1 cu. ft. = 7.4805 U. S. gal.

h = Depth of Liquid Inches	d = Diameter of Tank, Inches									
	12		18		24		30		36	
	Gal.	Cu. Ft.	Gal.	Cu. Ft.	Gal.	Cu. Ft.	Gal.	Cu. Ft.	Gal.	Cu. Ft.
2	0.64	0.0860	0.80	0.1072	0.93	0.1244	1.05	0.1400	1.15	0.154
4	1.73	0.2317	2.18	0.2920	2.57	0.3440	2.90	0.3878	3.21	0.429
6	2.94	0.3927	3.85	0.5149	4.59	0.6140	5.23	0.6988	5.80	0.775
8	4.14	0.5537	5.67	0.7578	6.85	0.9152	7.85	1.049	8.75	1.17
10	5.23	0.6994	7.55	1.009	9.26	1.238	10.72	1.432	12.0	1.60
12	5.87	0.7854	9.38	1.252	11.75	1.571	13.72	1.833	15.4	2.03
14	.....	.....	11.04	1.476	14.24	1.903	16.82	2.248	19.0	2.54
16	.....	.....	12.43	1.659	16.65	2.226	19.90	2.660	22.6	3.02
18	.....	.....	13.22	1.767	18.91	2.527	23.00	3.075	26.4	3.53
20	.....	.....	.....	.....	20.93	2.797	26.00	3.476	29.6	3.95
22	.....	.....	.....	.....	22.57	3.017	28.85	3.859	33.4	4.46
24	.....	.....	.....	.....	23.50	3.141o	31.49	4.209	37.4	5.00
26	.....	.....	.....	.....	.....	.....	33.82	4.521	40.4	5.40
28	.....	.....	.....	.....	.....	.....	35.67	4.768	43.7	5.84
30	.....	.....	.....	.....	.....	.....	36.72	4.908	46.6	6.23
32	.....	.....	.....	.....	.....	.....	.....	.....	49.1	6.55
34	.....	.....	.....	.....	.....	.....	.....	.....	51.2	6.85
36	.....	.....	.....	.....	.....	.....	.....	.....	52.9	7.07

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OPERATING ENGINES ON SEWAGE SLUDGE GAS*

Some Comments on Their Care and Operation

By THOMAS BROWN and GEORGE PRESTON

METHODS of sewage treatment, in which sludge is digested, made possible the production and connection of power producing gas from digesting sludge. In the earlier period of digestion of sewage sludge, the gas generated in the process was considered of no value, and it was burned to waste to prevent obnoxious odors. Then economy demands directed engineers to utilize the sludge gas in heating the sludge in the digestion tanks so as to maintain the required uniform digestion temperature, thereby improving the process of sludge digestion and the quality of digested sludge. In the past few years sludge gas has been put to use also in the generation of power by passing it from the digestion tanks through gas engines—these gas engines driving electric generators or air blowers or raw sewage pumps. To effect further utilization of the heat contained in the sludge gas, the jacket cooling water of the gas engines is passed through heat evchangers which heat the water circulated through the heating coils of the digestion tanks and building spaces. Additional heat is being recovered from the sludge gas by passing the engine exhaust through waste gas boilers to add additional thermal units to the hot water in the heat exchanger.

The Power in Sewage Sludge Gas

Gas from digesting sewage sludge is an excellent fuel for gas engines. It has an average heat value of 600 British thermal units per cubic foot, and values up to 650 or more units are obtained. Its composition is given in general as 64 per cent methane, 30 per cent carbon dioxide, 5 per cent nitrogen, a trace of hydrogen sulphide, and 1 per cent other gases .The gas is produced in a fairly uniform volume at all times in the normal operation of a sewage treatment plant; and possible peak flows or excess quantities may be stored in gas holders.

Generally a municipal sewage treatment plant of proper design can produce one cubic foot of gas per person in 24 hours; but an average conservative figure is three-fifths of a cubic foot. Supposing a sewage treatment plant serves a city of 100,000 people, and the average value of the sludge gas generated is 600 British thermal units per cubic foot, and assuming that three-fifths of a cubic foot of gas is generated per person per day, then the brake horsepower developed is: $(100{,}000 \times 3/5 \times 600)$ $(24 \times 10{,}000)$ or 150 horsepower. If one cubic foot of gas is generated per person per day, the horsepower generated would be 250 for a population of 100,000. If garbage is ground into the sewage, the gas production might be doubled, giving a power production of 500 horsepower per 100,000 population, or 5 per 1,000.

Scrubbing Sludge Gas

If the sludge gas contains an excessive quantity of H_2S or hydrogen sulphide, the pistons and cylinder walls of the engine are likely to be harmed; for the products of combustion, namely, the water and sulphur dioxide (SO_2), unite to form the very corrosive sulphurous acid (H_2SO_3); but any amounts of volatile sulphide are just as objectionable in the engine fuel as is hydrogen sulphide.

If the content of sulphide in sludge gas should exceed 0.07 per cent by volume, or about 50 grains in 100 cubic feet of gas, a gas scrubber should be placed ahead of the gas engine. The sulphur in the state of hydrogen sulphide is removed in the scrubber by passing the sludge gas through a bed of finely divided iron oxide held in wood shavings. The iron oxide reacts with the hydrogen sulphide to form water and sulphide of iron in accordance with the equation $Fe_2O_3 + 3H_2S = Fe_2S_3 + 3H_2O$. The wood shavings serve only as a sponge for the iron, keeping open the mass of iron oxide, so the gas can pass through the scrubber with a minimum of resistance. In order to promote the chemical action between hydrogen sulphide and iron oxide, the temperature of the gas scrubber should be kept above 60 degrees Fahr. Artificial heat may have to be supplied.

The gas scrubber may be built readily by the operating staff of the sewage treatment plant. If 160,000 cubic feet of gas is to be scrubbed per day, containing about 100 grains of hydrogen sulphide in 100 cubic feet of gas, the scrubber should be figured on the following basis: Enough iron sponge should be provided to last about 60 days. There is about 20 pounds of ferric iron in one cubic foot of sponge. Theoretically, 20 pounds or one cubic foot of ferric iron sponge will react with 127,000 grains of hydrogen sulphide. In actual operation, this reaction capacity will be cut to about one-third, or to 40,000 grains of hydrogen sulphide. Figuring 100 grains of hydrogen sulphide in 100 cubic feet of gas, and 160,000 cubic feet of gas per day, the total hydrogen sulphide to be removed in 60 days is 9,600,000 grains. Therefore the amount of iron sponge required is 240 cubic feet. The gas scrubber tanks should be large enough to hold this amount of iron sponge, and in addition there should be at least three inches of space at the bottom of the scrubber, underneath a tray, this space aiding in better distribution of the incoming gas to the sponge bed. The scrubber should be coated inside with a bituminous paint.

The iron sponge can be used over again after regeneration. When it has become exhausted or sulphided in the 60-day period, it is exposed carefully to the air so as to allow the iron sulphide to absorb oxygen, enabling the iron to revert back to its original ferric oxide state according to the equation $2Fe_2S_3 + 3O_2 = 2Fe_2O_3 + 6S$. There is so much heat generated in this reaction that there is danger of the iron sponge starting to burn and be destroyed. The rate of heat generation can be controlled by wetting the exhausted iron sponge thoroughly with water, taking it then from the scrubber tank and piling it in the open air. After the sponge has charged from blacw to brown in color, it can be placed back in the scrubber; it should be placed uniformly and tamped lightly as it is placed to fill all space evenly. The iron sponge can be regenerated two or three times in this manner, before it has to be discarded finally.

Engines for Sludge Gas

Gas engines for running on sludge gas are built generally in sizes from 15 to 1,800 horsepower. They are connected direct to electric generators, or to air blowers, or to sewage pumps by way of gearing, or to other mechanical equipment. They are used full or part time, all or part of the installation, as gas becomes available. For running a gas engine full load and full time in spite of insufficient sludge gas supply, city gas can be made available for standby.

The heavy duty slow speed gas engine is preferred to other gas engines. It can run for several years continuously with low cost of maintenance and repairs. In the smaller size of gas engine the valve-in-head type is preferred. By installing a carburetor, gasoline may be used as fuel instead of gas. The gas engine can be expected to have a normal life of 20 years. Repairs will run about $2.00 per year per brake horsepower. The gas engine is required to run almost continually, for if it is shut down, generally considerable electric power must be purchased and a high kilovolt-ampere demand results.

The sludge gas enters the engine by way of a gas mixing valve in which the gas is mixed with atmospheric air in proper proportion. The valve has a function which is similar to that of the carburetor used in gasoline engines, and it contains an air valve and a gas valve, each regulated separately; due to this arrangement the fuel mixture can be changed to suit the changing British thermal unit content of the gas. Just ahead of the gas mixing valve, the gas pressure is reduced to atmospheric pressure by use of a suitable pressure regulator. By installing a gas reservoir between the pressure regulator and the mixing valve, the operator can maintain a constant supply of gas at the atmospheric pressure; and this reservoir should be at least 30 cubic inches in size per horsepower of engine rating.

Generally speaking, at full load the fuel consumption of the gas engine does not exceed 10,000 British thermal units per brake horsepower-hour, which equals 15,000 British thermal units per kilowatt hour. If the sludge gas has a low heat value of 575 British thermal units per cubic foot, a kilowatt-hour generated requires 26.1 cubic feet of gas at full load, 27.5 at 5/6th load, 29.3 at ⅔rd load, and 32.7 at ½ load.

Installations of Gas Engines

Hundreds of installations of gas engines running on sewage sludge gas have been made during the last few years, including

*This article, from *The International Engineer*, so well tells the story of power generation from sludge digestion gas, that we have asked permission to reproduce it.—Ed.

at least 3,500 horsepower at Tallmans Island plant, New York City; 1,820 at the Coney Island plant, New York City; 835 at Peoria, Ill.; 360 at Detroit, Mich.; 300 at Cedar Rapids, Iowa. [See appended list of engine installations. The largest will be that at New York's Jamaica plant, with 3 engines of 1,440 H.P. each—i. e., 4,320 total H.P.—Ed.]

Gas and Air System.—The fuel gas system of the engine consists as a rule of a gas holder or reservoir (which may be a floating cover on the digester tank), a gas meter, a pressure regulator, a mixing valve, and a governor. The governor needs careful attention; its rotating parts must be kept lubricated properly—when the lubricating oil is discharged back to the crank case, attention must be given that the blowby gases are not depositing water in the governor. Keep all flat wearing surfaces of the governor free of depressions so that it continues to respond smoothly and rapidly. Inspect the governor every time the piston is removed.

Draw all intake air through adequate filters; for dust drawn into the engine with the air accelerates the wear of moving parts. All intake air lines should be large which prevents high losses in pressure when the engine is running at full load. The mixing valves need adjustment and lubrication occasionally. Water should be kept out of the gas piping.

Air Starting and Safety Devices

The air starting system of the gas engine includes air compressor, air reservoir tanks, and air piping. The air compressor should be inspected occasionally and repaired if necessary. See that the condensate drains properly from the air starting tanks and piping. Check blow-off cocks, and float-operated condensate separators on the tanks, periodically to be sure they remove water from the air storage tanks and air lines, and prevent the fouling of spark plugs due to injection of water into the engine cylinder.

The ignition system consists of magnetos, or battery-operated spark coils, timed distributors, high tension distribution wiring and spark plugs. Keep the voltage generated at a minimum to keep the magneto or spark coil in best operating condition; do this by keeping the spark gaps at or near the minimum for production of a good spark. Change or clean spark plugs and re-adjust the gap when the gap increases from the normal 0.025-inch setting to about 0.050 inch. The double electrode plugs are well suited to the gas engine. Spark plug gap adjustment may be required once a month. Clean magneto and distributors once each year and adjust or replace electrodes as needed. Check timing settings against the flywheel markings every time the timing mechanism is dismantled. Inspect high tension wiring for cracks and take precautions against possible cross firing where wires are adjacent to each other.

The engine protective devices generally are instruments actuated by pressure, temperature or mechanical movements, thereby operating electrical switches or relays. These relays and switches ground the magnetos and close the fuel gas valve, so that the engine is shut down. If the gas engine drives an electric generator, there is also a generator breaker which is tripped and the electrical system cleared for normal operation.

Pressure gauges, tachometers, pyrometers and other indicating devices should be kept in good operating condition. All delicate indicating devices and safety devices, and all gauges should be kept mounted on panels which are isolated from the engine so they will not wear due to vibration. The safety devices and gauges should be inspected and checked quite often to make sure they operate properly. Check the protective devices which shut the engine down when the temperature becomes excessive or when the pressure of cooling water or lubricating oil should drop below normal. Also check the overspeed protection device.

Maintenance Schedule

Following a routine maintenance schedule will insure the operation of the gas engine with a minimum of shutdowns. Details of maintenance on the gas engine are affected by the size, speed, operating condition and design of the unit. The operating engineer should have a routine maintenance schedule for each engine to follow throughout the year. He should keep a complete record of all maintenance work on the unit. Such records will help to work out a more reliable maintenance schedule and overhauling schedule as the years pass; and it enables the engineer to keep a closer check on the performance of each engine part. Dated records should be kept on crankshaft allignment, clearance of piston rings, bearing clearance. The operating engineer should take note also of the condition of piston rings, pistons, liners, oil conditions, dimensions and replacements of piston rings, diameters of liners, etc.

Main Crankshaft and Connecting Rod Bearings

After the gas engine has been run for some time, the main crankshaft and connecting rod bearings should be inspected at the rate of once per year. The bearings should be checked for clearance. The main crankshaft bearings must be checked for proper alignment; this should be done after the main bearings are adjusted and whenever the pistons are removed. Any misalignment can be determined by measuring the deflection of the crankshaft flanges when rotating the shaft. Mount a sensitive deflection gauge between the parallel crank faces of the crankshaft and take readings at 90-degree intervals in the shaft rotation.

If the gas engine is run with misaligned main bearings, a repeated flexing of the crankshaft metal occurs and in the end there will be crystallization and failure of the shaft. If the measured deflection of a large gas engine is .003 inches, the bearings should be adjusted. The bearing adjustment can be done by removing and scraping the lower half of the main bearings until the proper alignment has been obtained. To estimate the clearance in bearings, the most satisfactory way is to remove the small shims, reassemble the bearing until it just binds in the shaft, and then reinsert the proper size of shim to provide the correct clearance.

Every two or three years the clearance between the wrist pin and wrist pin bushings should be measured. If there is a thump of pronounced intensity, it indicates that a wrist pin assembly has become defective. In this case press the pin out of the piston, and measure the diameter of both the pin and bushing with a micrometer gauge. If excessive wear has occurred, grind the pins to true circular section and install new bushings of correct diameter.

Cylinders and Pistons

About every six to eight months the piston should be removed from the gas engine so that carbon deposits can be removed from around the rings and so that any rings showing excessive wear can be replaced. Clean the piston and piston rings thoroughly of gumming oil and hard carbon. The oil holes back of the piston grooves should be cleaned. Replace the piston rings on the same piston from which they were removed. Stagger the ring gaps on opposite sides of the piston. Liner packing rings should be checked for tightness after the engine has been reassembled, by placing the water jacket under water pressure and watching for water leaks into the crankcase.

If pistons are operated for a prolonged time with piston rings stuck, the piston and liner may become scuffed or gouged. If there is an indication that the piston rings are not working properly, clean the piston and free the piston rings at the earliest hour possible. When excessive blowby of cylinder gases into the crankcase is noticed, or when there is a marked increase in the consumption of lubricating oil, it is likely that some of the piston rings have become stuck in the piston ring grooves. By removing the piston rings promptly and correcting the condition, excessive wear on pistons, piston rings and cylinder liners resulting from higher temperatures and poor lubrication is stopped or prevented.

When the piston is being cleaned, inspect the piston rings for wear and ring gap. After the piston rings have been placed on the piston, check the clearance between the rings and groove, by use of a feeler gauge. The seat for the piston ring should be square, and there should be no excessive taper in the ring groove, for this may lead to sticking of the rings. If power or compression rings are to be used over again, chamfer them slightly with a file to remove sharp wearing edges. The oil control rings of the scraper ring type should have a relatively small bearing area and also a sharp scraping edge.

It is best, after the head has been taken off and the liner and piston removed, to measure the diameter of the liner in both transverse and lateral direction at three points of the piston stroke. Then make a record of the readings and compute the amount and rate of wear. After a new engine has been run for some time, the average maximum increase in diameter of the liner may amount to from .0004 to .0005 inches in 1,000 hours of operation. Cylinder liners may stand three reborings, after which new ones should be installed. Any rings showing excessive ring gap, lack of tension and an undue reduction in radial thickness should *not* be used again.

Valves Need Checking

Whenever the engine is dismantled, check the air starting valves and the valve motion devices including gears, camshaft, cams, push rods, and valve operating rockers.

Reseat the exhaust and intake valves periodically to be sure

they will work properly. The exhaust valves should be reseated every two to three months, and the intake valves should be reseated every four to six months. If the valve discs and seats have become pitted, they should be placed in a lathe and ground with a carborundum wheel until all pitting is removed; or the pitting may be removed with a cutting wheel.

Small quantities of carbon and dust depositing between valve stems and guides may cause the valve to stick. This occurs most likely after the engine has been shut down. Clean the valve surfaces and inspect them for proper clearance. Excessive clearance of the valve stem will cause the valve to move sideways too much so that it may not close completely over the entire seat; and the valve disc may start to warp in reference to the stem. It is possible that the valve disc has warped already on the stem, and therefore will have to be aligned. If the valve guides have worn too much, cast iron bushings can be inserted to provide again a proper clearance. Hard steel bushing inserts may be obtained for the valve cages, and also replaceable valve seats.

In larger engines the valves should be in removable cages, or they should be installed in some other way which makes them easily removable for inspection and for grinding of valves. The operating staff should keep a stock of removable valve seat inserts.

If the valve does not seat properly, injury may result to the valve and the gas engine will lose power and operate unevenly. After the valves have again been assembled in their cages, the seating surface should be lapped in with a good grinding paste. Later the fit should be checked by using a colored paste, as for example prussian blue. First apply some prussian blue paste to the cage seat. Then wipe to a thin film with a dry cloth. Then place the valve in the cage. Then rotate the valve disc slightly. Then check the disc which should show a continuous ring of blue. If it does not, more grinding is necessary to make a true contact. Smaller valves may be ground completely more economically by the lapping method only.

Having replaced reground valves, the clearance between the valve stem cap and the valve operating mechanism should be adjusted properly. Inspect this clearance quite often for about two days after they have been put in service and adjust whenever necessary; for there may be a change in clearance for a day or two due to the hammering of the valve. It is a good plan to *inspect the valves about once a week* to make sure that a proper clearance is maintained.

Lubrication

Most careful attention should be given to the lubricating oil system consisting of engine sump or oil storage tank, oil pump, oil filter, or screen, oil cooler, oil centrifuge, or purifier, and oil distribution lines. The best quality of oil obtainable for the purpose should be used in the plant. After a new installation has been started, the oil should be checked say weekly for suspended solids, for viscosity, for water, for neutralization number, and also for sulphur content if the gas burned contains an appreciable amount of hydrogen sulphide. An oil changing and oil purification schedule should be worked out from experience to meet engine requirements; and once this has been done, the *oil conditions* may have to be *inspected only about once a month.*

When operating conditions are normal and a batch system of purifying the oil is used, the oil should be removed fully once a month and new or purified oil substituted. The oil tends to build up solids, due to thermal decomposition. Water which forms in the crankcase of the engine from blowby of combustion gases, leads to the forming of oil-water emulsions which are sometimes called sludge. When accumulating in the crankcase, this sludge should be removed completely before any new or purified oil is added to the system. The rate of oil decomposition and the formation of sludge being increased by excessive blowby of the combustion gases, oil inspection and oil changing may be required more frequently if excessive blowby occurs.

The operator should never allow the sludge to accumulate so it will be pumped through the oil circulating system. The oil pump for the main distribution system should be checked frequently to insure the correct discharge pressure. Oil screens installed to remove coarse particles should be cleaned every three or four months.

It is best to remove the water and sludge from the crankcase by using a gear pump with flexible suction hose. Do not wipe the crankcase with rags, for the lint left might be pumped through the oil lines and deposited on the screen, which will result in lower oil pressure in the system.

Oil Cooling and Purifying

The temperature of the oil in the cooler should not exceed 160 degrees Fahr., and water should not be allowed to leak into the oil. There should be thermometers in the oil line before and after the coolers. When the temperature differential for operating the cooler is not sufficient, the cooler should be bypassed and cleaned. The cooler should be cleaned periodically; remove the tube bundle and boil it in a 10 per cent solution of tri-sodium phosphate (which is an emulsifying agent) until the sludge settled in the tube bundle is removed and softened. Then blow the bundle out with compressed air to remove traces of softened material.

The oil in the circulating system should not contain more than 0.75 per cent suspended solids. The solids content can be reduced in a centrifuge to about 0.15 per cent, and the water content to less than 0.1 per cent moisture. For centrifuging, the oil should be heated first to 140 to 160 degrees Fahr., with a relatively low-temperature element. The oil heater should not be of the instantaneous type, for the element in this heater is so hot that there is danger of breaking down the oil, causing carbon deposits around the heater, which insulates the heater and causes it eventually to burn out.

When the engine is dismantled, check the oil distribution lines. Check also the link mechanism of the force feed lubricator for wear and replace worn parts if any. Also check the lubricating pump for rate of discharge.

Waste Heat Equipment

The cooling water should be maintained soft enough to prevent calcium and magnesium salts from depositing on the surface of the water jackets. The engine cooling water jackets should be inspected periodically for scale deposits. If there are heavy deposits, correct the water conditions to prevent further deposits. Excessive deposits overheat engine parts as a result of lowering the heat transfer rate. It is preferable to use a closed system of water cooling; for in the open system using evaporation by cooling tower, deposits are built up. When a closed cooling system is used, the return water to the engines can be kept between 90 and 100 degrees Fahr. and the water discharged from the engine jackets should not exceed 160 degrees Fahr. Periodically check the water pumps, water jackets and cooling facilities of the cooling water system.

All or part of the waste heat from the engine jacket cooling water can be recovered for use in the water heating system of the building spaces and digestion tanks. This heat recovery saves coal or oil fuel which would otherwise be required to supply the heating system. In a four-cycle slow-speed gas engine of about 90-horsepower capacity, for example, operating under full load, the heat content of gas fuel is applied as follows: 25.5 per cent in useful work, 3.5 per cent in radiation, 33.5 per cent in jacket cooling water, 37.5 per cent in exhaust and friction. Practically all of the 33.5 per cent of heat going to the jacket cooling water can be recovered in a hot water heating system.

It is recommended to *have the cooling water enter the engine jacket at between* 110 *and* 130 *degrees Fahr. and have it leave the engine at between* 140 *and* 160 *degrees Fahr.* To effect efficient cooling of the engine, about 3,300 British thermal units must be removed from the engine jacket water per brake horsepower per hour. In passing the engine jacket water through the heating coils of the digester, therefore, this amount of heat must be extracted from them both winter and summer; if not all of this amount of heat can be extracted from the so-called water cooler in the digester, (that is, the digester heating system), part of the jacket water has to be circulated through a separate water cooler, as for example during the summer season, to obtain the required cooling effect.

The heat that can be removed from the jacket water of a 90-horsepower engine under full load is 297,000 British thermal units per hour. For a 30-degree Fahr. temperature rise the amount of jacket cooling water required is 1,190 gallons per hour. Cooling of the water is effected in the digester heating system or in some other way, and a circulating pump of at least 1,190-gallons-per-hour capacity is needed to effect positive circulation of water. There should be a vented surge tank at the high point of the jacket water system, placed just ahead of the circulating pump or between the engine outlet and the cooler. There should be a thermometer at the cold water inlet and at the hot water outlet.

Waste Heat Boiler

In some gas engine installations the exhaust gases are exhausted to the atmosphere by way of exhaust silencers and waste heat boilers. If a waste heat boiler is used to extract enough heat from the exhaust gases to cool them from 900 degrees Fahr., to 300 degrees Fahr., and a high boiler water temperature is desired to guard against possible condensation and corrosion, the jacket cooling water of the engine may be pumped into the

boiler after preheated to 140 degrees Fahr. The waste heat boiler acts as a silencer for the engine, and if there is a by-pass around the waste gas boiler, it should have an exhaust silencer.

The waste heat boiler requires great care in operation. Water must not be permitted to condense on the gas side of the tubes if maintenance costs are to be kept low. Condensate forming on the gas side of the boiler tubes results in rapid corrosion and tube failure. When waste heat or exhaust gas boilers are not in use, they should be disconnected completely, for valves used in exhaust gas lines do not seat tightly and the gases filtering into the boiler corrode the gas side of the tubes due to deposit of condensate.

Condensation of water on the gas side of tubes, due to cold boilers, can be prevented by generating steam in the boiler and converting it to heating uses; or by heating water in the boiler and controlling its flow in such a manner that the temperature of the water will not drop below 180 degrees Fahr. At points where radiated heat will affect the operation of engine devices, the exhaust gas lines should be kept insulated. The heat radiated from the engine, which is about 3.5 per cent, cannot be transferred to the hot water heating system; this radiation helps cut down the heating load by heating the engine room partially.

The dewpoint, or the temperature at which condensation on the cooling surfaces of the waste heat boiler begins, is about 130 degrees Fahr. As the gas contacts the boiler surface, there is a sudden drop in temperature; therefore the exhaust gases can be cooled only to that temperature where they will leave the boiler without condensation. *The inlet temperature of the boiler water should not be less than* 130 *degrees Fahr.; and a safe minimum temperature of the exhaust gas is* 300 *degrees Fahr.*

If feed water is not available at 130 degrees Fahr., the waste heat boiler may be used to generate low-pressure steam which is passed to an independent tubular heater, the condensate returning to the waste heat boiler generally well above 130 degrees Fahr. Excess sludge gas, that is not used in the gas engine, can be fired in a gas boiler to supplement the waste heat recovery boiler.

Exhaust gases from a 4-cycle gas engine having water-cooled exhaust manifold, have a temperature of 800 to 900 degrees Fahr.; and exhaust gases from a two-cycle gas engine have a temperature of 300 to 500 degrees Fahr. A 90-horsepower four-cycle engine will produce about 1,130 pounds of exhaust gas per hour at full load, from which at least 150,000 British thermal units can be recovered per hour.

SLUDGE GAS ENGINE INSTALLATIONS IN THE UNITED STATES

Year	Location	No. of Units	Hp. Per Unit	Driving
1926	Plainfield, New Jersey	2	15	Blower
1928	Charlotte, North Carolina	1	225	Blower
	Charlotte, North Carolina	1	75	Pump
1930	Rockville Centre, New York	1	30	Blower
	San Bernardino, California	1	40	Generator
1932	Rockville Centre, New York	1	60	Blower
	Springfield, Illinois	1	180	Blower
1933	Fort Atkinson, Wisconsin	1	40	Generator
1934	Springfield, Illinois	1	157	Bl.&Gen.
	Menominee Falls, Wisconsin	1	5	Generator
1935	Ann Arbor, Michigan	1	120	Blower
	Burlington, Wisconsin	1	15	Generator
	Cedar Rapids, Iowa	1	210	Generator
	Durham, North Carolina	2	150	Generator
	Greencastle, Indiana	1	50	Blower
	Los Angeles County, California	1	200	Pump
	Madison, Wisconsin	1	230	Blower
	Monroe, Michigan	1	25	Pump
	Ontario, California	1	40	Blower
	Palo Alto, California	1	40	Generator
	Peoria, Illinois	1	535	Blower
	Peoria, Illinois	1	300	Blower
	Toledo, Ohio	1	300	Pump
	Toledo, Ohio	1	225	Generator
1936–37	Atlanta, Georgia			
	Intrenchment Creek	2	120	Generator
	Clayton	2	180	Generator
	Aurora, Illinois	2	75	Pump
	Battle Creek, Michigan	1	50	Pump
	Cheboygan, Michigan	2	120	Pump
	Davenport, Iowa	1	75	Pump
	Davenport, Iowa	1	75	Generator
	District of Columbia	1	1200	Generator
	Dixon, Illinois	1	30	Generator
	Edwardsville, Illinois	1	15	Generator
	Elmhurst, Illinois	1	30	Generator
	Grand Rapids, Michigan	1	50	Generator
	Green Bay, Wisconsin	1	120	Generator
	Green Bay, Wisconsin	1	90	Generator
	Hutchinson, Kansas	1	90	Generator
	Janesville, Wisconsin	1	30	Generator
	Kewanee, Illinois	1	30	Generator
	Kokomo, Indiana	1	45	Generator
	La Crosse, Wisconsin	2	90	Generator
	La Porte, Indiana	1	27	Generator
	Litchfield, Illinois	1	30	Generator
	Madison, Wisconsin	1	230	Generator
	Michigan City, Indiana	1	75	Blower
	Monroe, Wisconsin	1	30	Generator
	New York, New York—			
	Coney Island	3	300	Generator
	Oshkosh, Wisconsin	2	160	Generator
	Richmond, Indiana	1	120	Blower
	State College, Pennsylvania	1	15	Generator
	Topeka, Kansas	2	180	Blower
	Wichita, Kansas	1	75	Pump
1938	Albuquerque, New Mexico	1	150	Generator
	Cedar Rapids, Iowa	1	100	Pump
	Chicago Heights, Illinois	1	100	Blower
	Crawfordsville, Indiana	1	44	Blower
	Columbus, Ohio	3	750	Generator
	Detroit, Michigan	1	364	Generator
	Findlay, Ohio	1	90	Blower
	Fort Dodge, Iowa	1	275	Generator
	Greensboro, South Carolina	2	160	Generator
	Lake Charles, Louisiana	1	50	Blower
	Lansing, Michigan	1	240	Blower
	Michigan City, Indiana	1	74	Generator
	Newcastle, Indiana	1	45	Blower
	Okoboji, Iowa	2	40	Blower
	Pueblo, Colorado	3	75	Pump
	Wooster, Ohio	1	15	Pump
1939	Carlsbad, New Mexico	2	50	Pump
	Crawfordsville, Wisconsin	1	44	Pump
	Escanaba, Michigan	1	40	Generator
	Ithaca, New York	1	50	Pump
	Kenosha, Wisconsin	2	75	Pump
	Manitowoc, Wisconsin	1	75	Pump
	Manitowoc, Wisconsin	1	35	Pump
	Medford, Wisconsin	1	15	Generator
	Menominee, Michigan	1	19	Pump
	Midland, Michigan	1	15	Pump
	New York, New York—			
	Tallmans Island	2	800	Blower
	Tallmans Island	2	450	Blower
	Tallmans Island	2	350	Pump
	Tallmans Island	2	150	Pump
	Rock Island, Illinois	5	125	Pump
	Rock Island, Illinois	1	75	Generator
	Rock Island, Illinois	2	75	Pump
	Two Rivers, Wisconsin	1	43	Blower
	Two Rivers, Wisconsin	1	40	Pump
	Wausau, Wisconsin	1	75	Generator
	Wausau, Wisconsin	1	50	Generator
	Wausau, Wisconsin	1	50	Pump
1940	Adrian, Michigan	1	45	Blower
	Aurora, Illinois	2	50	Generator
	Belleville, Illinois	1	100	Blower
	Butler, Pennsylvania	1	80	Generator
	Cedar Rapids, Iowa	1	100	Pump
	Cleveland, Ohio	2	390	Blower
	Fort Wayne, Indiana	1	450	Blower
	Galesburg, Illinois	1	40	Pump
	Gary, Indiana	3	175	Pump
	Gary, Indiana	2	300	Blower
	Greenville, South Carolina	2	225	Generator
	Marion, Indiana	2	75	Blower
	Marion, Indiana	1	50	Pump
	Metairie, Louisiana	1	50	Blower
	New York, New York—			
	Jamaica	3	1450	Generator
	Coney Island	3	920	Generator
	Peru, Indiana	1	45	Blower
	Pontiac, Michigan	1	150	Blower
	Quincy, Illinois	2	44	Pump
	Shawano, Wisconsin	1	35	Generator
	Sharon, Pennsylvania	2	80	Generator
	South St. Paul, Minnesota	1	125	Pump
	Springfield, Missouri	2	198	Generator
	Stevens Point, Wisconsin	1	75	Generator
	Wisconsin Rapids, Wisconsin	1	75	Generator
1941	Carbondale, Illinois	1	28	Blower
	New Iberia, Louisiana	1	36	Blower
	New York, New York—			
	Bowery Bay	2	800	Generator
	Rahway, New Jersey	2	50	Pump
	San Angelo, Texas	1	20	Pump
	San Dimos, California	1	250	Pump
	Washington, D. C.	1	1200	Generator
1942	Columbus, Ohio	3	775	Generator
	El Paso, Texas	2	210	Generator
	Leesville, Louisiana	1	40	Blower
	Los Angeles County, California	2	110	Pump
	San Leandro, California	1	110	?
	Toledo, Ohio	1	310	Pump
	Toledo, Ohio	1	180	Generator
1943	Battle Creek, Michigan	1	50	Pump
	Fort Huacho, Arizona	3	1000	?
	Fort Huacho, Arizona	1	750	?
	Muncie, Indiana	1	50	Pump
1944–45	New York, New York—			
	26th Ward—Under contract	3	1300	Generator
	Owls Head—Under design	6	1250	Generator
	Newtown Creek—Under design	5	1450	Generator
	Hunt's Point—Under design	2	920	Blower
	Hunt's Point—Under design	2	920	Bl. or Gn.

GAS ENGINE PERFORMANCE

Accomplishments at Marion, Ind.

By DAVID BACKMEYER
Supt., Water Dept. and Sewage Works, Marion, Ind.

THE sewage treatment and garbage disposal plant at Marion, Ind., was put in operation in July of 1940. The plant is of the activated sludge type with separate sludge digestion. Two digestion tanks, each 65 ft. in diameter and equipped with Pacific Flush Tank Co. floating covers, provide a total digestion capacity of 167,000 cu. ft. With the exception of one district in the southern part of the city, which is served by a pumping station, all the sewage flows to the plant by gravity. The present resident population of the city is 30,000, and the average sewage volume pumped in 1943 was 3.82 mgd.

The power requirements of the plant fall under the following four general heads: (1) pumping of the raw sewage, a lift of 33 ft. from the wet well to the grit channel, (2) providing compressed air for the activated sludge plant, (3) operation of vacuum filter equipment for removal of digested sludge, and (4) many small motors driving sludge collectors, comminutors, water pumps, sludge pumps, water circulating pumps, etc. The average electric power consumption was 14,990 kwhr. per month in 1943.

Power Plant

Three Climax "Blue Streak" gas engines have been in service since the plant was first started about four years ago. Two of these engines, rated at 75 horsepower, are direct connected to 1800 cfm. Roots Connersville blowers, and the third, rated at 50 horsepower, is direct connected to a 4000 gal. per min. Chicago sewage pump.

Gas production, augmented by the addition of ground garbage to the sewage sludge digesters, has been sufficient to furnish heat for all the buildings and digestion tanks and, at the same time, provide through the means of the gas engines 73 per cent of the total power requirement of the plant. Natural gas, purchased from the private utility company as a stand-by fuel, is piped to each of the three gas engines. By simply turning two valves and adjusting one carburetor screw, the fuel supply may be changed from sewage gas to the 1000 Btu. natural gas, and vice versa.

Heat from the engine cooling jackets and from the exhaust gases is recovered by heat exchangers and used for heating the digesters and plant buildings. Two cooling coils submerged in the aeration tanks provide cooling water temperature control in the summer, when excess heat must be dissipated. All three of the engines are protected by automatic shut-off switches which operate on low oil pressure and/or high water temperature. Automatically controlled signal horns are located in strategic parts of the buildings to warn the plant operators when either of these conditions exist. Electrically driven stand-by pumping equipment, automatically controlled from float switches, is provided for emergency use in case the gas engine unit develops mechanical trouble.

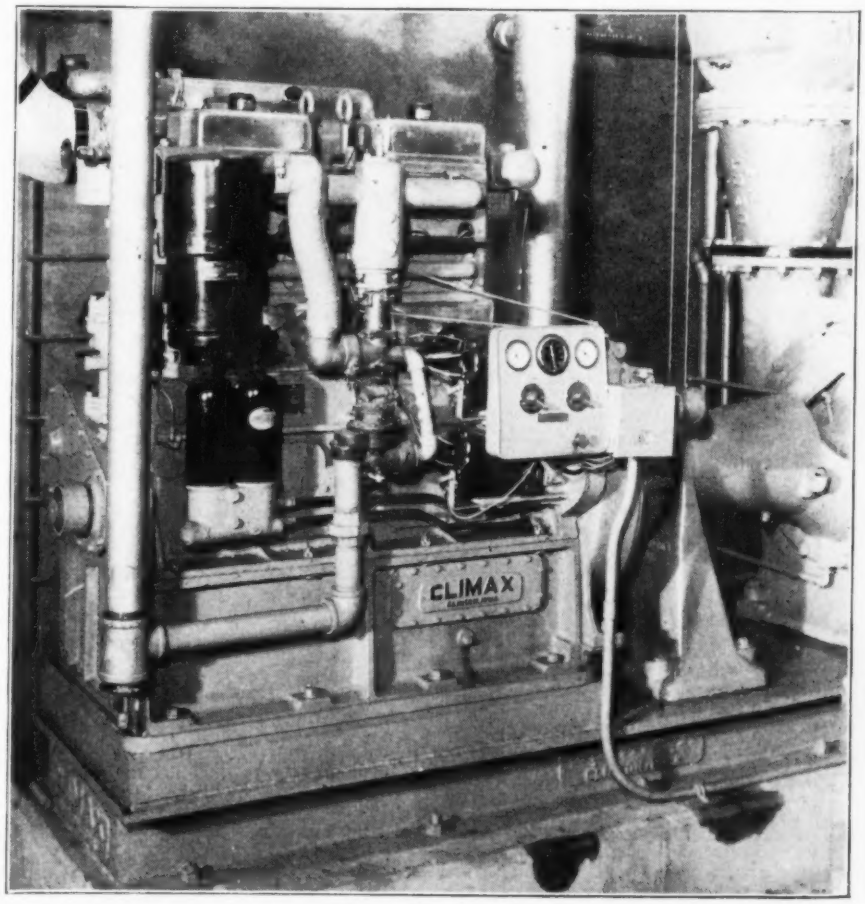
Engine Performance

Inasmuch as the pump engine has operated a greater number of hours than either of the blower engines, its performance record will be reviewed in some detail first. This engine has pumped more than 99 per cent of the sewage treated at the plant to date. It is a four-cylinder Climax engine, having a six-inch bore and seven-inch stroke with a total piston displacement of 792 cu. in. The engine has a rated horsepower of 57.6 with a maximum horsepower of 100 at top speed of 1000 rpm. The crankcase has a capacity of sixteen gallons. Since sewage gas is relatively dry hydrocarbon fuel, a Marvel Mystery inverse oiler is installed to lubricate the upper end of the valve guides and the top edge of the pistons. An industrial oil of S.A.E. No. 35 has been used exclusively in the crankcase. This oil costs 51c per gallon and is purchased from the Gulf Oil Co. A Honan-Crane continuous type oil filter keeps the oil clean and clear at all times. A Gulf Oil product known as Gem Oil "C" is bought at 42c per gallon and has been found to be very satisfactory for use in the inverse top oiler.

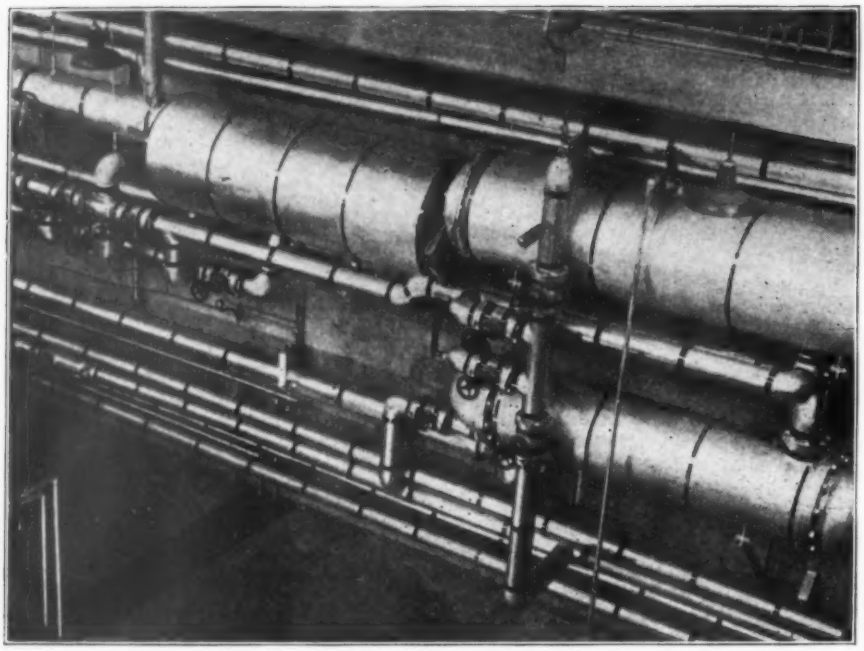
Driving the pump by direct connection, the speed of the engine is governed by a float in the raw sewage wet well. Its operation at different speeds for varying sewage flows is, therefore, practically automatic. Since the pump is most efficient at discharge rates close to 4000 gal. per minute, the engine is capable of handling peak flows at times as high as 6.0 mgd.

Maintenance Practice

A maintenance policy, based upon regular periodic inspections of the engines, has been followed. The length of time an engine is operated prior to dismantling and servicing depends upon the condition of the engine after a certain number of hours of continuous operation. Our pump engine was first operated for 8830 hours before it was shut down for overhauling. This period was found to be too long. The piston tops and engine heads were badly carboned; the compression rings were stuck and the oil rings were in such bad condition that they could not possibly function properly. The next inspection was, therefore, made



50-HP. Climax Engine Direct Connected to Chicago Sewage Pump



Muffler and Two Heat Exchangers for Recovery of Gas Engine Heat

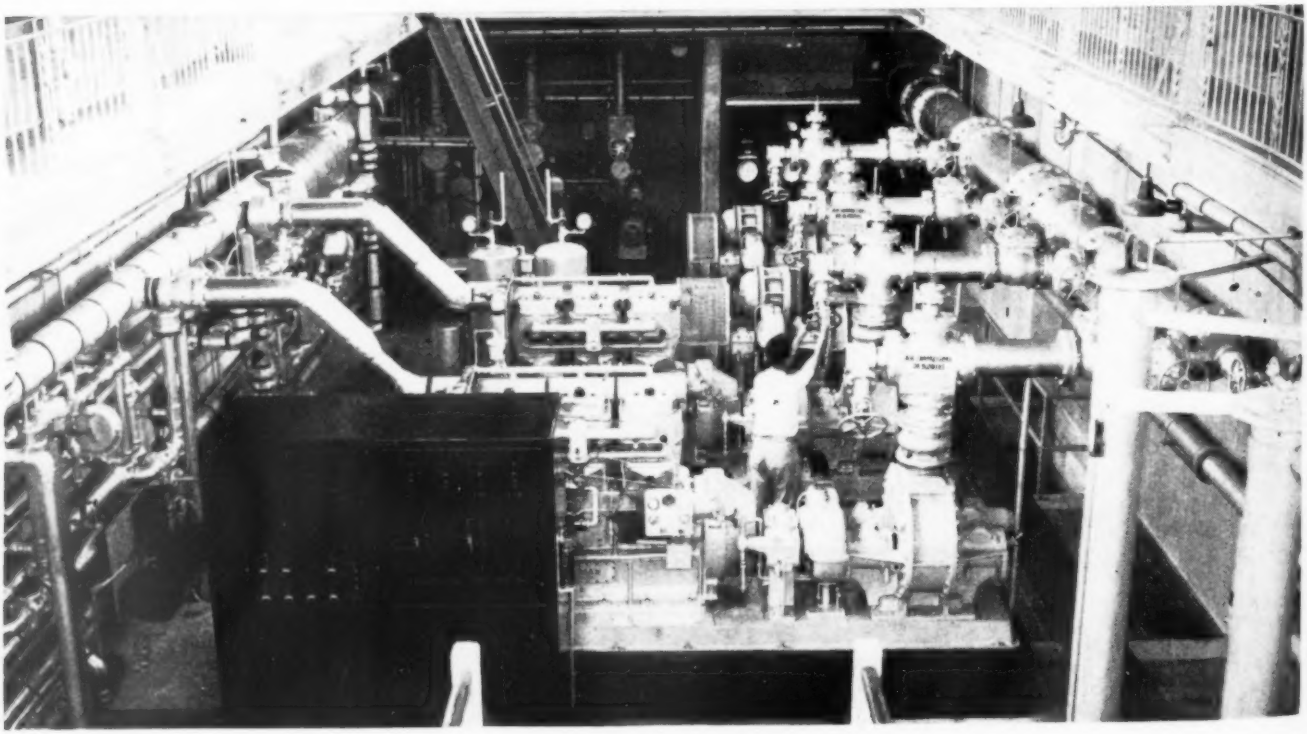
after a continuous run of only 5130 hours. This inspection found the engine in much better condition. The rings were heavily carboned, but were still free and not frozen tight in the grooves as was the case at the time of the previous overhauling. The third overhaul was made after another run of 4310 hours, the fourth after 4720 hours, and the fifth after a run of 5530 hours. It has never been necessary to make a major repair of any kind on this engine in between our regular servicing periods.

To dismantle the small 50-hp. engine and thoroughly overhaul and inspect it requires about 36 man-hours' labor. This work is done by our own plant operators and, in most cases, the engine is not shut down for more than 48 hours when the work is being done. The three engines are insured against mechanical failure of parts while in operation. This insurance covers labor and parts needed for replacement in case of mechanical breakdown. The insurance company sends a trained inspection engineer to the plant to look over the engine each time it is serviced. In accordance with the terms of the insurance contract, recommendations of the inspector as to maintenance policies are carried out by the insured. The insurance cost for the pump engine is $53 per year. We believe this expense is not out of line with the service and protection rendered.

Typical Inspection Report

The following inspection report, made on January 6, 1944, is typical and will give an idea as to what repairs were needed to put the engine in first-class mechanical condition after a run of 5330 hours and a total operation period of 28,544 hours: Engine shut down and completely dismantled for overhauling. Very little carbon found on piston tops and in heads. Engine could have run another month, as all piston rings were free, although oil rings were showing signs of sticking. All three main bearings were opened and one 0.003-inch shim was removed from each side. Rod bearings were carefully inspected and found to be in good condition. All piston rings were replaced by new rings (std. 6-in. by ¼-in. rings used) and all valve springs were replaced by new springs. All valves were replaced by valves that had been re-faced by machining. New valve-seat insert sleeves were put in both intake and exhaust for the first time. All connecting rod bolts were replaced with new bolts (per recommendation of insurance company) as the old bolts had seen more than 25,000 hours of operation. No. 1 cylinder wall showed slight "score" mark from cause unknown. Other three cylinder walls in good shape. Cylinder walls were "miked" for wear as follows:

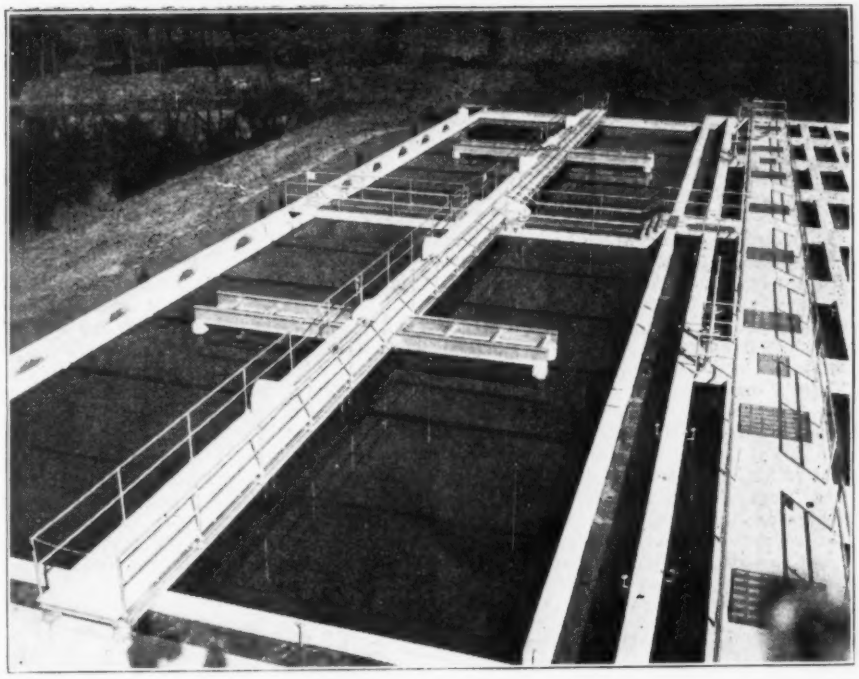
	2-in. down	4-in. down	7-in. down	bottom
Cylinder No. 1	0.0055	0.0033	0.0020	0.000
2	0.0079	0.0060	0.0039	0.000
3	0.0050	0.0031	0.0021	0.000
4	0.0055	0.0040	0.0021	0.000



Power House, Marion, Ind., Sewage Treatment Works
(Twin 75-HP. Climax Gas Engines Direct Connected to 1800 cfm. Roots-Connersville Blowers)

Spark plugs were cleaned and adjusted, and a complete set of new gaskets were used in assembling engine. Crankcase cleaned and flushed—very little sludge found in bottom. New earth cartridge put in Cranite filter, and crankcase filled with new oil. Oil consumption was a little less than one quart per day when engine was shut down for inspection. Oil is used as long as 6000 hours and Cranite filter is changed at 2500 hours.

All of our "Blue Streak" engines are provided with dual ignition systems. American Bosch magnetos are furnished for both the four- and six-cylinder engines. During the first year of operation we had considerable trouble with our magnetos. Only in a very few instances would these units stay on duty for a continuous run of 90 days. Repair costs were quite high, as gear-rotor assemblies were usually needed when an overhauling job was done. At the suggestion of the local American Bosch service headquarters, we purchased one utility magneto for the four-cylinder engine and one for the two six-cylinder engines. We were then able to remove magnetos at regular intervals of sixty days and maintain a strict servicing schedule. Our records show that from August, 1940, to May, 1944, the maintenance cost of the three magnetos used on the pump engine was $207.93, fifty per cent of which was for labor. This gives an average monthly cost of $4.84 for the 43 months operated. The total maintenance cost for the five magnetos used on the larger engines was $293.97, fifty per cent of this total also being for labor. These engines have collectively operated 58 months, giving average monthly cost of $5.05.



Final Settling Tanks, Marion, Ind. (Aeration Tanks on Right)

Dollar Savings

In preparing an estimate of the dollar savings effected by the pump engine, as compared to electrically powered equipment, an operation period of 31,238 hours ending March 31, 1944, was used. The maintenance cost for a seven-month operation period from August, 1941, to March, 1942, was as follows:

Crankcase oil—71 gal.	$ 30.53
Top oil for valves—90 qts.	9.00
Grease for water pump	.50
Re-facing valves	1.60
New piston rings	11.00
Magneto repairs	35.00
Cranite filter cartridges	3.00
Hose and gaskets	5.00
Two sets spark plugs	6.40
Complete set valve springs	5.20
Extra labor—garage mechanic	16.00
Insurance (pro-rated)	31.00
Total	$154.23

To completely overhaul this engine in January, 1944, as previously described, cost a total of $180.15, including insurance charge. Therefore it would be fair to assume the maintenance cost for an average year to be $286.60. (Note—No charge is made for gas purchased from the utility company since this item is more than offset by the value of the heat recovered from the engines.)

The electric utility company agreed to furnish current to the plant at the rate of 1.16c per kwhr., provided the plant was made "all electric" and that no gas engines were installed. To duplicate the work done by the gas engine in pumping the raw sewage would require an estimated 191,000 kwhr. in a twelve-month year. This amount of current would cost $2215.60 at the best electric rates available. The net saving for the pump engine in one year's time is, therefore, $1929.60.

The two larger engines (rated at 75 hp.) driving the air blowers have operated just as efficiently as has the smaller engine. The record of their maintenance cost is parallel to that of the 50-hp. engine in all respects. Due to the variance of the B.O.D. load handled at the plant during the prolonged rainy periods, two blowers are not needed throughout the entire year. Our operation record shows that the blower engines were operated singly for 77 per cent of the time and together 23 per cent of the time during an average year. The average maintenance expense, based on a 5930-hour operation year, for the No. 3 engine amounted to $245.76. The No. 4 engine ran a few less hours, but, since it required the same number of servicings, the cost is also figured at $245.76. The total cost for the blower engines is, therefore, $491.52.

To duplicate the work done by the blower engines in one year would have required an estimated 546,000 kwhr. of electric current. This electricity would have cost $6333.60. The actual savings accomplished by the two engines in furnishing all the air for the activated plant is, therefore, $5842.08. In one year's time the three engines have earned a total of $7771.68. In four years' operation time they will have earned $31,086.72. It is obvious that Marion's city officials made a wise choice in choosing gas engines in preference to electric powered equipment when the plant was being designed.

Some Suggestions

Before concluding, consideration should be given to a few of the mechanical devices and design features that have helped to make our gas engine operation experience a pleasant one to date. Following are some of the more important items:

(1) Engines located in building that has *ample* ventilation.

(2) Plenty of working space around engines so that dismantling is an easy job for the operator.

(3) Provide a tool room and workshop, with cabinet space for stocking replacement parts.

(4) Provide control signals to warn operators when engines are in trouble.

(5) Have a reliable source of stand-by fuel so that engines can be kept in operation continuously.

(6) Install heat recovery facilities of sufficient size so that engine temperatures may be kept constant by automatically controlled valves.

(7) Provide oil filters to keep lubrication at highest possible efficiency.

(8) Have sufficient number of thermometers located in water circulation system so operators can easily check temperature conditions.

(9) Wherever possible, provide duplicate engine-driven equipment for emergency use.

(10) Speed of pump engine should be controlled automatically by float tube from sewage well.

Although no definite post-war plans for further gas engine facilities have been drawn up as yet, it is entirely possible that the Marion plant may be entirely free from all outside power sources at some future date. Sewage gas production approaches the 90,000 cu. ft. per day mark in the summer months, when considerable garbage is being digested. Some days as much as 25,000 cu. ft. of gas is actually wasted. This would point to engine-generator sets being installed with purchased utility gas filling in when sewage gas production does not meet the requirements of the engines. Further operation economies would undoubtedly be accomplished if such a procedure were properly worked out.

THE "MUD-HOG"

An Air-Lift Subaqueous Trencher Which May Also Be Used for Handling Sludges, Sand, and the Like

By HARRY U. FULLER
Chief Engr., Portland Water District, Portland, Me.

BECAUSE of war conditions in Portland, Maine, in 1943 it became necessary to devise a method of digging a trench 500 feet long in 30 feet of water without the use of the customary bucket on a derrick.

The material in the harbor bottom is a plastic blue clay and the method adopted was to cut the clay with water jets and remove it by air-lift of the heavy clay suspension.

The principle of the air-lift pump is that a column of water in a vertical pipe when full of air bubbles is lighter than the water on the outside of the pipe, and the unbalanced force causes the column of air-water mixture in the pipe to rise as long as air is forced into the pipe.

On page 146 of the Reference and Data Section in *Water Works and Sewerage* for May, 1937, the principle of pumping by air-lift is effectively explained in detail.

The air-lift trencher has two elements—the water jets and the air-lift pump. These were built into one unit, which could be easily handled and suspended from the boom of a lighter.

Construction of Trencher

The air-lift trencher (see cut) consisted of lengths of 8-in. pipe screwed together, reaching from the harbor bottom to a few feet above the deck of the lighter, about 40 feet.

On the upper end was an 8-in. quarter bend with a 3-ft. piece of 8-in. pipe screwed into it to throw the mud away from the lighter. The lower section of the 8-in. pipe was 5 ft. long and around it, 6 in. from the bottom, was welded a 16-in. steel drum 8 in. high. The drum was sealed top and bottom with ring plates. A 2-in. air pipe was attached to the drum through the top ring plate. Through the pipe within the drum were five equally spaced 11/16-in. holes for admission of air from the drum to the lift pipe.

On the other side of the 8-in. lift pipe was the 3-in. water jet pipe with 1-in. tees at 8-in. intervals. Into each tee was screwed a 1-in. one-eighth bend and into the bend a 4-in. long nipple, flattened at the outer end to leave ¼-in. wide opening to produce flat cutting jets.

The water pipe was carried by fittings around the 8-in. pipe under the drum with three jets pointing downward. The three pipes, from the lower section to the top of the trencher, were held together by welded braces.

The dimensions of the trencher were taken from an article in the *Engineering News Record* on June 5, 1941, telling how an air lift was used for cleaning mud and sand out of cofferdams.

The same proportions found to work successfully in the 12-in. lift in New York were used in building the tools herein described.

The areas of the air ports and air drums and the capacity of the air compressors were thus proportioned to the area of the air-lift pipes used on the Portland trenchers.

Three of these air-lift trenchers were built: An 8-in. pipe with 11/16-in. air ports inside the drum, requiring 400 cu. ft. of free air per minute. A 5-in. pipe with 7/16-in. air ports requiring 150 cu. ft. of free air per minute. A 4-in. pipe with 3/8-in. air ports, requiring 100 cu. ft. of free air per minute.

All three units worked very well with one-third less air than called for in the article referred to. However, with the full amount of air called for they worked much better.

When the proper amount of air is being applied, the flow is steady. If irregular flow or surging develops the amount of air must be readjusted. One outstanding advantage of this type of equipment for pumping mud, sludge, grit, etc., is the smooth interior waterway which permits anything which enters the lower end to continue to the outlet unobstructed.

Method of Use and Possible Applications

The trencher was hung from the side derrick boom on the lighter. Gasoline-driven air compressors of the required capacity were placed on the deck of the lighter and connected to the upper end of the 2-in. air pipe with air hose.

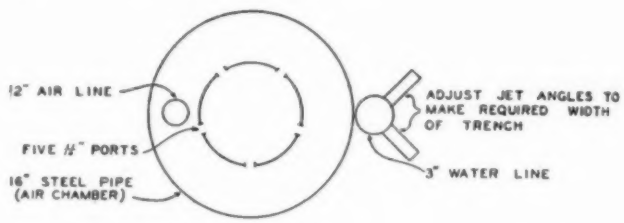
To provide water for the jets, a gasoline-driven fire pump, with a capacity of 500 gallons per minute at 100 pounds pressure, was also placed on the deck of the lighter and connected through fire hose to the upper end of the 3-in. water pipe. Because of the liberal diameter of the water pipe running to the foot of the trencher a large part of the 100 pounds pressure at the pump was available at the cutting and stirring jets.

The horizontal and vertical angles of the jets should be adjusted to give the required width of trench and to cut off the face of the clay and not merely bore holes in it.

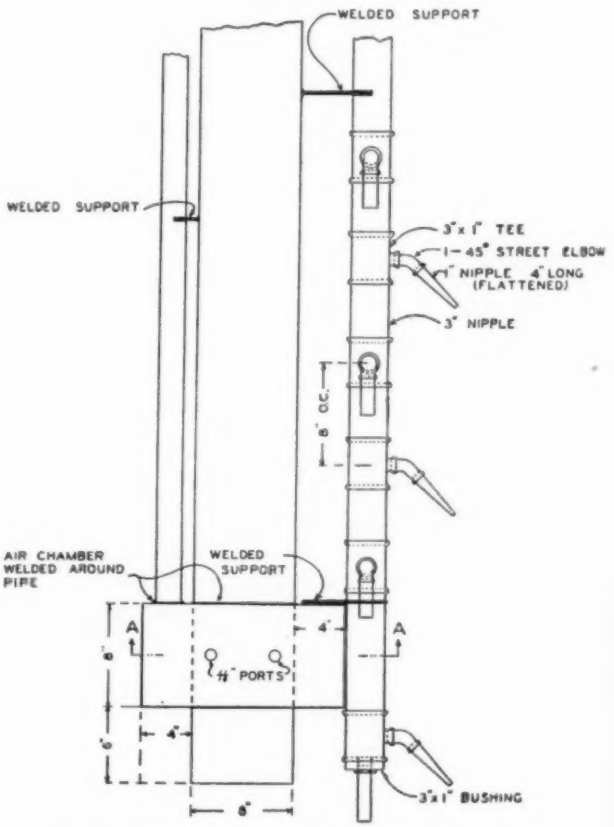
It was found to be helpful to keep the trencher constantly moving up and down a few inches. This was probably because the jets were some inches apart. The rate of trenching varied with the quality of the material. In medium soft clay the speed reached 25 feet per hour of actual digging and the trench on the harbor bottom was about three feet wide at the top.

It is believed that an air-lift device of this type will have a variety of uses in water and sewage works practice in addition to trenching. For instance, these units might be used for lifting (and at the same time washing) sand and small gravel. A small unit may be used for removing sand from rapid sand filters and transferring it from one unit to another. It could be used in excavating, desludging basins or lagoons.

In sewage it could be used to transfer sludge, lift and wash grit, de-sludge tanks or lagoons, and like uses. The units are readily portable, with a short riser and air pipe subject to extension to any desired length through use of plain end pipe lengths—and gasketted flexible couplings.



SECTION A–A



Detailed Sketch of the 8-in. P.W.D. Air-Lift Trencher

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DIAPHRAGM AND PLUNGER TYPE LIQUID PROPORTIONING PUMPS.
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MANUALLY ADJUSTABLE AND AUTOMATIC PROPORTIONAL UNITS.
DESIGNS FOR HIGH OR LOW PRESSURE.

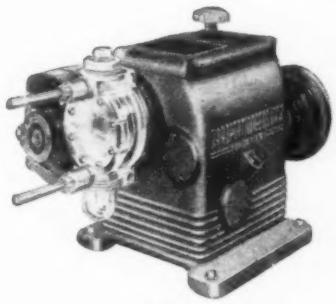


Fig. 1
Heavy Duty Chem-O-Feeder
0-8 gal./hr.
0-100 lbs./sq. in.

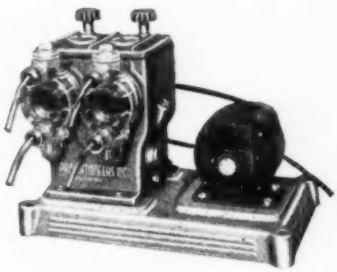


Fig. 2
Duplex Chem-O-Feeder
0-16 gal./hr.
0-100 lbs./sq. in.

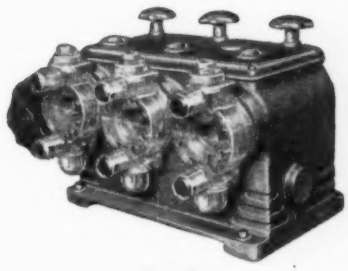


Fig. 3
Triplex Chem-O-Feeder
0-24 gal./hr.
0-100 lbs./sq. in.

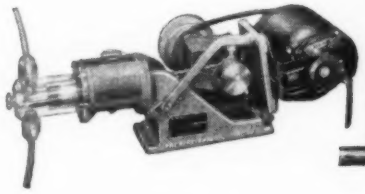


Fig. 4
High Pressure Midget
Chlor-O-Feeder
0-6 gal./hr.
30-200 lbs./sq. in.

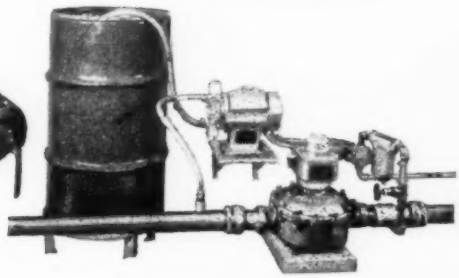


Fig. 5
Low Pressure Automatic and
Proportional Chem-O-Feeder
0-5 gal./hr.
4-100 lbs./sq. in.

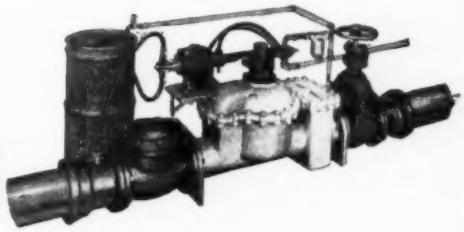
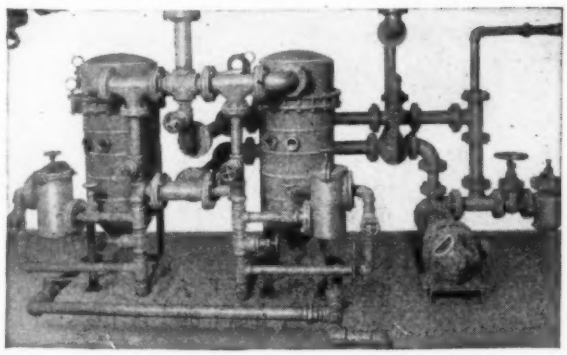


Fig. 6
High Pressure Automatic and
Proportional Chem-O-Feeder
0-5 gal./hr.
30-200 lbs./sq. in.



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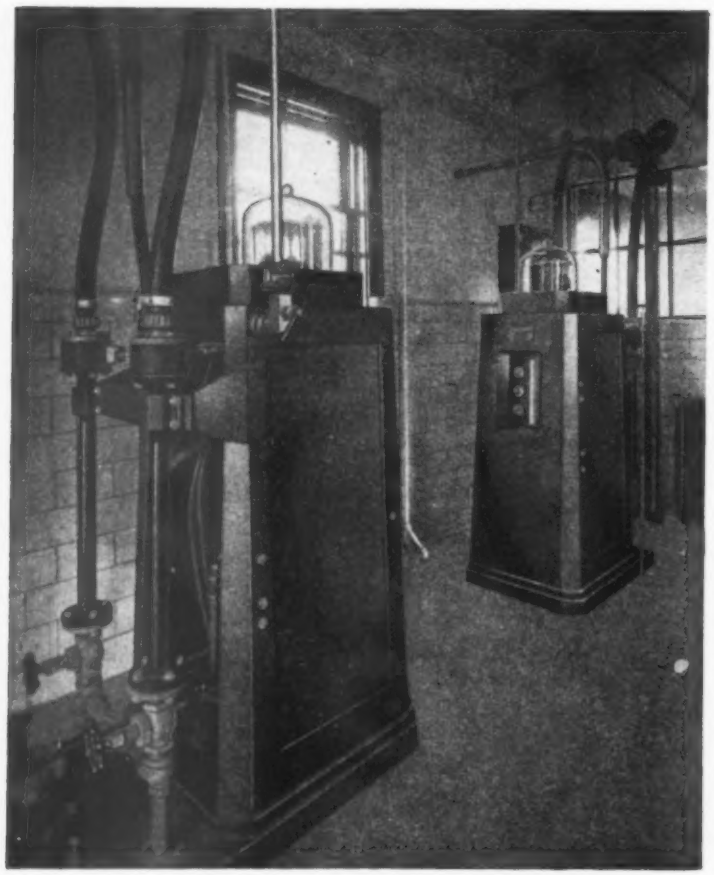

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ODOR CONTROL

The control of obnoxious odors from sewage treatment works, long outfall sewers and other disposal plant appurtenances is distinctly a field for chlorination. Neutralization of hydrogen sulfide (the gas responsible for these obnoxious odors), or prevention of its formation by controlled application of chlorine, is not only quite practicable but also results in economy of operation due to reduction of deterioration in interceptors and allied appurtenances.

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Treatment plants discharging effluents into water courses having extreme seasonal variation in flow have profited from controlled chlorination to reduce biochemical oxygen demand. Field tests have often shown a reduction in the five day B.O.D. of up to 30 per cent.

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W&T representatives will gladly discuss any specific sewage chlorination problem. Technical bulletins on the many phases of sewage chlorination and W&T Chlorinators will be sent free on request.

ODOR CONTROL AND CONTROL OF FILTER FLIES

A Review of Current Practices

By A. A. HIRSCH
State Department of Education,
Baton Rouge, La.

THESE subjects, control of odors and filter flies, are associated because trickling filter units are usually involved, and because certain of the control measures are of value with both difficulties. Whereas odors may occur in any type of plant, filter flies are usually, but not necessarily, specific to trickling filters.

Odor Control

Causes of Odors

Not considered here are odors due to overloading of a receiving stream with raw or settled sewage, or introducing untreated sewage into an otherwise adequate body of water, but without proper provision for mixing and carry-off. For the latter case, an intercepting sewer, installed to move the outfall to a more remote or turbulent location, frequently solves the problem.

Broadly classified, odors may arise from either:

(a) Receipt of stale or septic sewage at the treatment plant;

(b) Failure of treatment units to perform properly; or

(c) A combination of both causes (a) and (b).

Septic sewage emanates odors of decomposition in which that of hydrogen sulfide predominates, mingled with ammonia, amines, mercaptans, and indol and skatol derived from decomposition of proteins. Tallowy odors form from changes in fats.[1] Hydrogen sulfide, detectable at 0.001 ppm. concentration in air, is produced by the bacterial splitting of complex sulfur-containing compounds, and by the anaerobic reduction of sulfates, attributed mainly to *Vibrio desulfuricans*.[2] Odors are more prevalent in hard water regions where sulfates are high, or when sulfur-containing wastes are disposed of through the sewers. Hydrogen sulfide is also the principal toxic component of "sewer gas," one part in 20,000 of air causing marked irritation to eye and respiratory membranes.[3] In the collection system, hydrogen sulfide also destroys mortar joints and concrete pipe, particularly above the flow line where oxidation and moisture form sulfuric acid in dilute solution. In addition, manhole steps and cast iron manhole covers are destroyed in the same manner. At the plant, the efficiency of primary sedimentation is reduced, the liquor is harder to treat and is liable to release odors on the filters. Production of hydrogen sulfide in septic sewage continues through the primary settling tank, especially so when this unit is excessively large for the sewage flow. A simple test for the presence of hydrogen sulfide in a sewer atmosphere consists in exposing a strip of filter or blotting paper moistened with a 5 per cent lead acetate solution; darkening indicates a positive test. Lacking lead acetate, a silver coin rubbed bright will turn brown where hydrogen sulfide exists to a degree which will cause odor nuisance or create acid attack.

Operating troubles that are liable to develop odor conditions are generally those that interfere with primary sedimentation or with percolation through the filters. Since septic sewage settles poorly in the primary tank, suspended solids are carried over to the filter where they cover over the zoogleal film, or plug the stones to form pools of quiescent liquor that ferments on standing. Overloading of primary tanks or a low rate filter frequently leads to ponding of the filters. In the Imhoff tank, deposits allowed to remain on the sloping floor, or too high a sludge level, reaching into the slot, release bubbles in the settling compartment, thereby interfering with sedimentation. Sprinkling of septic liquor through the atmosphere releases odors, particularly on hot, humid, still nights when temperature promotes decomposition and motionless air conditions cause the odor to hug the ground. Occasionally, the filter bed itself is the source of ponding, when too fine a filter medium is provided, or when the fines are all dumped in a few areas, or when stones are broken by careless placement. Sand beds will hold ponds when overloaded, or when not raked over the surface frequently enough to break up the top deposits.

Other minor sources of odor in the plant are from accumulations of screenings or unwashed grit deposits, and from discharge and wetting by rain on beds of poorly digested sludge. Odors from waste digester gases are avoided by burning in a field torch, or discharging through a high-level stack. When high buildings are nearby, chlorination of the gases prevent complaint from odors.[4]

Physical Control of Odors in Sewers

More advantageously prevented than removed, the three C's of odor control have been stated as Care, Cleanliness and Chlorination. Prevention is concerned with the early arrival of fresh sewage at the treatment plant through clean, well-sloped lines, and the orderly management of the primary tank to permit unhampered sedimentation, and of the filter unit to prevent ponding.

A Trickling Filter Which Is Beginning to Show Signs of Ponding in Right Foreground.

Flow of water at a manhole does not indicate a clean sewer; even a fire plug discharge soon levels out to merely a fast-moving stream in the sewer. This flow may flush along loose materials, but revolving jets or blades are needed to scour the walls clean. Roots are removed by means of a helical cutting auger.

Physical Control of Odors at the Plant

Overcapacity of primary tanks may be reduced by cutting unnecessary units out of service, or by dilution, through recirculation of the filter effluent.

Pooling over the filter surface may be corrected mechanically by such methods as providing proper sedimentation, forking the stones to an 8- to 12-inch depth, poking through ponds with a slicing bar, hosing with a heavy stream from a fire hose, withdrawing the filter from service for one or two days, or by flooding for three to four days. Ponded areas of stone may be dried out for several days by closing off individual distribution laterals, or by plugging sections of the revolving arm. Distribution piping and underdrains should be flushed out when possible. Weeds and other vegetation should not be permitted to grow on the filter bed. Localized overdosing of certain areas on the bed, sometimes a cause of pooling, may be determined by means of a pan test in which a row of cans is lined radially outward from the center-post, and their catchment measured after several revolutions of the distributor. From the relative volumes of liquor, each square foot of surface should be dosed alike, the nozzles being adjusted to spread the flow evenly. Operating nozzles should be kept cleared.

Presence of filter flies below the density that causes nuisance has been noted to clear ponded areas[5, 6] by the voracious feeding habits of the larvae on the biological film. On the other hand, many filters are in operation without ponding and with flies practically absent.

Chemical Control of Odors in Sewers

Chlorination up-sewer, in subresidual dosage, prevents growth of sulfate-splitting bacteria and postpones decomposition of organic material, thereby controlling formation of hydrogen sulfide in the sewers. A four-day retention test with raw sewage showed that at least 20 to 25 per cent of the chlorine demand must be satisfied to suppress sulfides to below 1 ppm.[7] Due to poor mixing, sludge deposits, and side wall slimes, a notably higher dosage, generally around 10 ppm., but ranging from 3 to 30 ppm. to satisfy up to 50 per cent of the chlorine demand, is required in the sewer lines.[8] If hydrogen sulfide has already been formed, much more chlorine is required to react in chemically equivalent quantity, namely, 2.1 ppm. of chlorine per 1 ppm. of hydrogen sulfide removed. A portable chlorinator is useful for this operation, and its programming should be based on a survey for both hydrogen sulfide and residual chlorine. Chlorination also reduces the 5-day B.O.D. about 2 ppm. for every part per million of chlorine absorbed.

Copper sulfate has been found valuable for controlling micro-organisms, such as *Beggiatoa*,[9] which also is thought to produce hydrogen sulfide from sulfates, as sulfur granules are visible in the cell contents. Roots in the sewer are killed by flushing through 3 pounds of this chemical from a house toilet. A complete flush should be provided, as the concentrated solution is corrosive to iron pipe. For keeping sewage fairly fresh, 2 pounds is applied through a manhole, during low flows to conserve chemical. Sewage from lines so treated is reported to reach the disposal plant without change in pH-alkalinity balance.

Ferrous chloride, produced by passing a chlorinator discharge upwards through a contact tower filled with scrap iron, removes hydrogen sulfide by direct combination chemically; it is especially useful when the hydrogen sulfide concentration is already high.

Other methods[10] for odor control include:

(a) Aeration, to furnish oxygen and to inhibit anaerobic decomposition, particularly that by the sulfate-reducing organisms. Pre-aeration, 0.05 cu. ft. per gal. of sewage, for grease removal also aids in preserving sewage fresh.

(b) Dilution, to furnish oxygen, to reduce concentration of solids, or to speed flow, particularly in partly clogged pipes. Dilution may be obtained from nearby stream or canal water, or from air-conditioning effluents. Use of an automatic dosing siphon connected to the potable water supply is a dangerous practice, allowing possibility of back-siphonage.

(c) Temperatures below 70° F., to prevent growth of sulfate-reducing micro-organisms.

(d) Chilean salt-petre, to furnish oxygen through chemical reaction and thereby prevent breakdown of sulfates.

Chemical Treatment for Odors at the Plant

Inasmuch as most trickling filter units are provided with chlorinators, 3 to 6 ppm. dosage in the wet well or the dosing tank is used to relieve septic action in the primary settling tank or to reduce release of odor at the filter nozzles. Application of dry hypochlorite, about 10 pounds per 1,000 sq. ft. filter area, directly on the bed is widely used to correct ponding by sloughing off some of the biological film. Caustic soda, sprinkled over the surface of a clogged filter, removes deposited grease, forming soluble soaps which are washed away. A 55-ft. filter was so treated with 150 pounds of flake caustic.[11]

Chemical coagulation, as with ferrous chloride or ferric chloride, made locally by passing chlorine water over scrap iron and tin cans in a ceramic Scott-Darcy tower, removes odor by adsorption and by direct action with hydrogen sulfide.

Chlorinated hydrocarbons, such as Chloroben,[12] dripped into the trunk sewer in advance of the wet well, inhibits the organisms responsible for septic conditions in the Imhoff settling compartment, and so prevents formation of odors in these units. Activated carbon, 3 to 5 ppm. dosage, has corrected odors, during summer, in septic primary and secondary settling tanks.[13] At another plant, 17 to 25 ppm. of carbon applied through a homemade feeder to the primary tank influent reduced the odor satisfactorily.[14] For trapping odors during transfers of sludge into tankers for dumping at sea, the Wards Island, N. Y., plant passes displaced air through a dust filter and then through cannisters of activated carbon made from coconut shells.[15]

Lime treatment of sewage reduces odor due to hydrogen sulfide, but increases ammoniacal odors.[16] Ozone, introduced into the body of the sewage, and ozone-laden air, contacting the surface layer, have been proposed for sterilization and deodorization.[17] In two recent symposia[18, 19] are reported the odor experiences of nearly 30 contributors; a later article surveys odor-reduction practice at various plants in New York state.[20]

Filter Flies

Two species of moth-fly cause nuisance in percolating filter plants, *Psychoda alternata*, in hot and warm countries, and *Psychoda severeni*, in cooler climates. From egg to adult stage for development of the fly varies between 10 days to 3 weeks; according to season and temperature. Eggs are deposited in the moist film; pupae and larvae grow in the interstices between the stones. Larvae move about like worms in the slime, submerging their heads while searching for food and breathing through the tail; they can survive submergence for some time. The protected under-surface of the stones become covered with mature insects. The body of the adult fly is gray-brown, with long, transparent wings. Their dense hairs present resistance to flight and restrict travel. Flies are found also around unprotected latrines and open gutters receiving sewage. Methods for their reduction or elimination may be by biological, mechanical or chemical means.

Biological Control

A mixed micro-fauna, containing forms more carnivorous than psychoda, has been recommended for biological control.[21] *Archorutus viaticus*, a small, blue-black, wingless jumping insect, keeps filters relatively free of flies. Presence of frogs and snails is noted by several operators to be helpful.

Flooding

In filters having solid side-walls, flooding for 12- to 24-hour periods at intervals for two weeks wash out larvae and eggs, with slight temporary reduction in efficiency of the biological film. If flooded at night, some of the adult flies are entrapped in their hiding places and are drowned. Where flooding is impossible, controlled drying, as for ponding, may be beneficial.

High-Rate Filtration

Flies are said generally not to breed on recirculating filters[22] over which the influent is continuously dosed. Unloading of the film is continuous, thereby denying a stable location for the deposition of eggs. This unloading feature also controls ponding. Nevertheless, installations of this type are known to become infested and one even to increase breeding.[23] High-rate filters allow a monopoly by only one or two species, in contrast with a more varied fauna in ordinary filters.[5]

Screening

Filters with open stone, piled along the angle of repose, or having open checker-work walls, by giving easier access to the adult fly, seem to favor breeding more than do those of solid brick or concrete wall construction. However, even completely enclosed filters have been noted not to prevent the spread of flies.[24] To screen effectively, the filter is best provided with artificial ventilation, the exhaust duct is screened, and such insects as are caught there sprayed with insecticide. High grass and weeds on the adjacent lawn should be mowed close to the ground to eliminate resting places.

Blow-Torch

Flames impinging over the tops of the filter stones destroy eggs and larvae near the surface of the bed, and kill adult flies that are hit.

Chlorination

A 3 to 5 ppm. dosage of chlorine in the applied settled sewage, every five to ten days, is effective, but better economy is obtained by spreading hypochlorite powder over the filter surface. Chlorination is best applied at night, during low flows, to conserve chemical. One method[23] consists in stopping the flow, spreading 100 pounds of hypochlorite per 500 sq. yd. of filter surface, dosing two minutes, then draining for 15 minutes. This procedure repeated three times abates flies for a week without appreciable detriment to biological activity of the filter.

Insecticides

Contact of various pyrethrum, rotenone, creosote, nicotine and many other insecticidal solutions with larvae and adult flies is effective. A kerosene solution containing ½ pound of pyrethrum flowers per gallon, occasionally stirred for 24 hours, makes a satisfactory spray.[25] Chloroben,[12] applied for odor control, also exhibits incidental larvaecidal action. Housefly pupae and larvae, frequently found in sludge drying on sand beds, have been killed by a spray of No. 2 Diesel oil.[26]

Especially conspicuous and promising is "D D T" (dichlorodiphenyltrichloroethane), a contact insecticide, generally stable to light and oxidation, and compatible with most of the other insecticidal ingredients. On account of its war-restricted distribution the possibilities of this chemical around the sewage plant are only beginning to be realized, and may provide for complete elimination of undesirable insects. At a Louisiana army camp, several pounds of "DDT" dissolved in kerosene and applied experimentally at the dosing tank to a large recirculating filter, killed all flies almost instantaneously, but caused a visibly increased unloading from the stone for several days. For best results the rotary filter arms and all other structures offering a resting place for adult flies were also sprayed. This chemical can be applied within plant buildings as a spray or as a brushed coating. Plywood panels, coated with an oil-bound paint containing 5 per cent DDT kills domestic flies.

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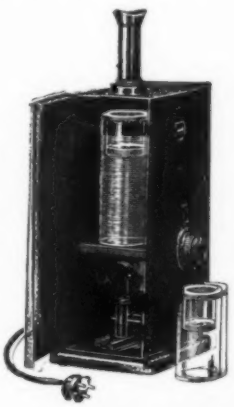

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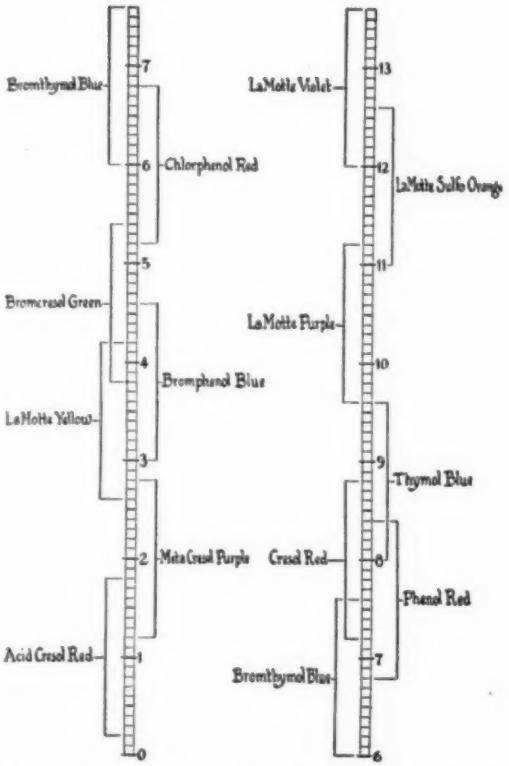
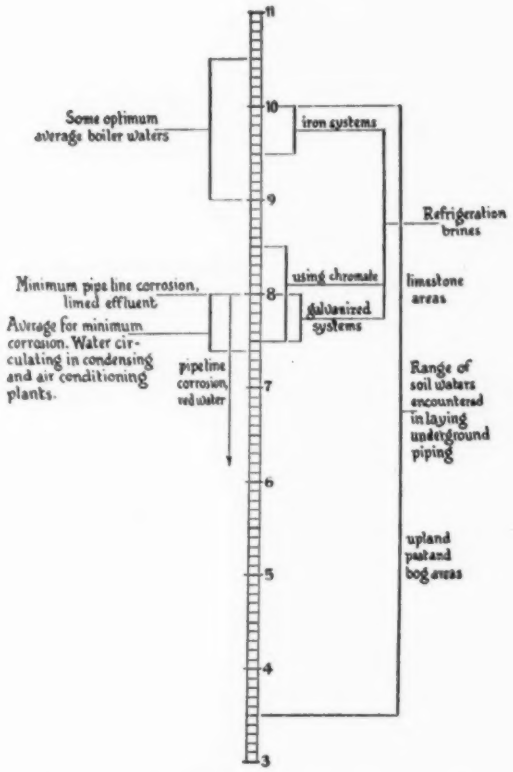


Ranges of Selected Indicators for pH Determination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Refrigeration brines
using chromate
limestone areas
galvanized systems
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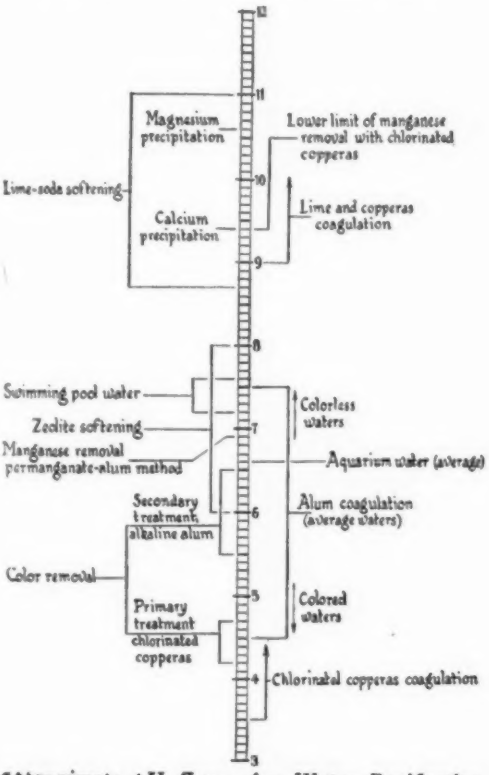
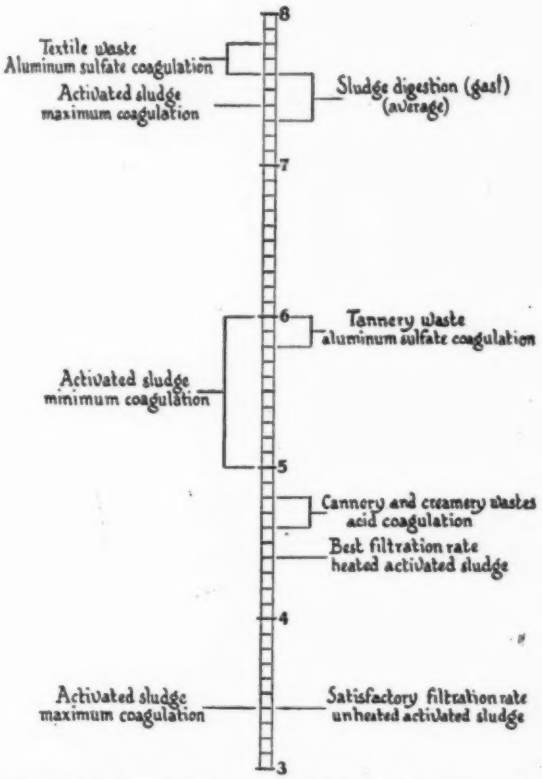
Average for minimum corrosion. Water circulating in condensing and air conditioning plants.
pipeline corrosion, red water
Range of soil waters encountered in laying underground piping
upland peat and bog areas

Approximate pH Zones Encountered in Common Corrosion Problems



Approximate pH Zones for Water Purification


Textile waste
Aluminum sulfate coagulation
Activated sludge maximum coagulation
Sludge digestion (gas!) (average)
Tannery waste aluminum sulfate coagulation
Activated sludge minimum coagulation
Cannery and creamery wastes acid coagulation
Best filtration rate heated activated sludge
Activated sludge maximum coagulation
Satisfactory filtration rate unheated activated sludge

Approximate pH Zones for Treatment of Sewages and Some Industrial Wastes

*Reproduced from the reference work "pH and Its Practical Application"—LaMotte, Kenny & Reed—(Williams and Wilkins, Publishers, Baltimore, Md.)

DELIVERY CAPACITY AND PRESSURE DROP THROUGH BRASS AND COPPER PIPE

This chart reveals the correct size of brass pipe or copper pipe or tubing to install for desired delivery of flow and residual pressure requirements.

Since the chart has been developed from reliable formulae, developed at Harvard University for straight runs of brass and copper pipes, it is necessary to allow something additional in the way of friction through valves, elbows and fittings. On this account, for the average service line it is good practice to add 20% to the actual length of run of pipe to compensate for loss of head through valves and fittings.

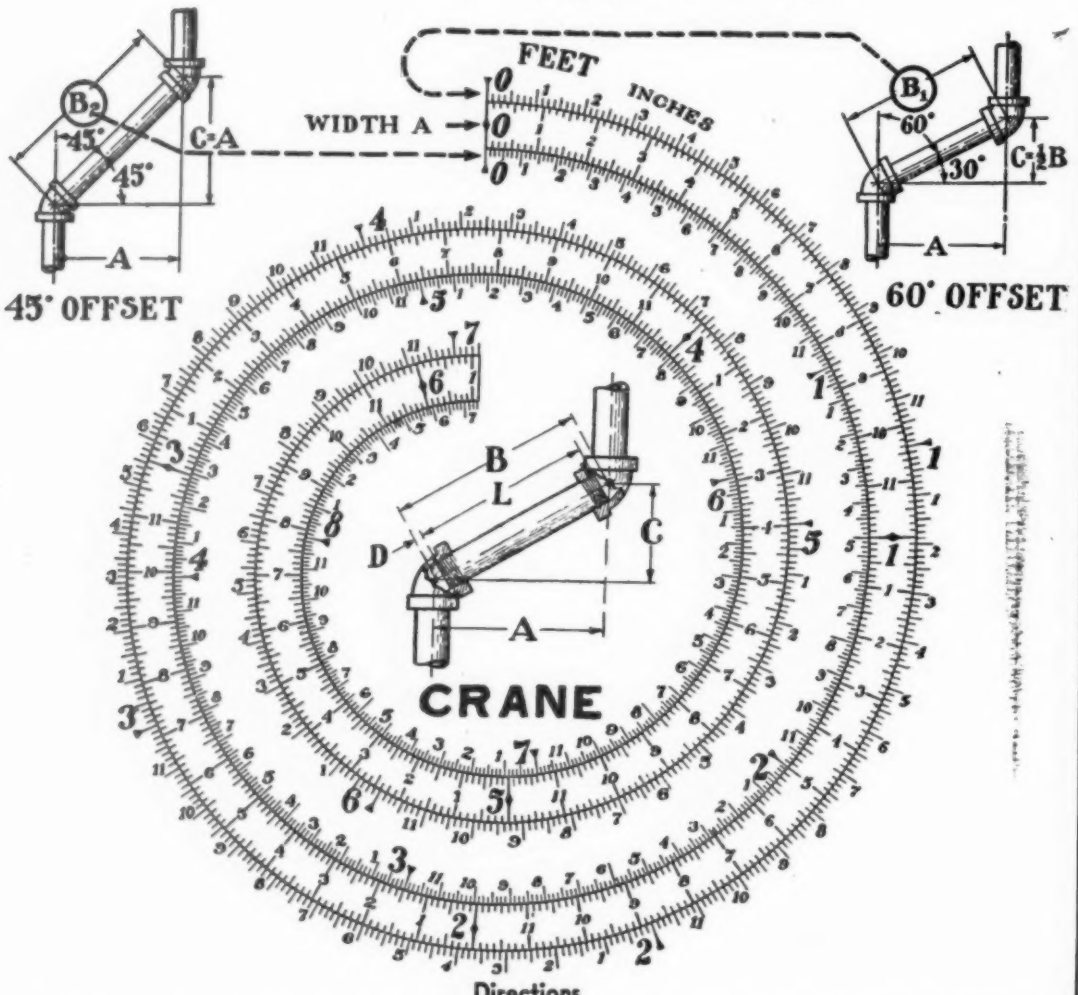
Example: There is an available pressure drop of 30 lbs., and the minimum delivery requirement at fixtures is 35 gals./ min. What size of pipe is required for this service?

Assume a run of 100 ft. to compensate for fittings, etc., add the 20% to secure an "indicated" run of 120 ft. From the 120 ft. point on the base line run up to intersect the down slanting line representing 1 in. pipe; then to the right to the 30 lb. up slanting pressure line; then up to the G.P.M. line. The answer is 36 gals./min. If the ¾ in. size had been tried the result would have been only 17 gals./min. delivery with a 30 lb. pressure available at the main. On the other hand a 1¼ in. line will deliver twice the requirement—i. e., 70 gals./min. Or, for the 35 gal. rate of delivery only 10 lbs. of pressure would be required.

It is evident, therefore, that the chart will reveal the proper size of pipe to deliver a required flow with any desired residual pressure at the outlet. In this case use the difference between the normal minimum pressure at the main and the residual pressure desired as the "available" pressure and proceed as in the example given.

OFFSET CHART FOR 45° & 60° SCREWED FITTINGS

(Copyrighted by Crane Co.)



Pipe size...	⅛"	¼"	⅜"	½"	¾"	1"	1¼"	1½"	2"	2½"	3"
45° deduct.	.....	¾"	⅞"	¾"	1"	1⅛"	1⅜"	1⅝"	2"	2"	2⅜"
60° deduct.	.....	.....	.....	1"	1⅛"	1⅜"	1⅝"	2"	2⅜"	2½"	3"
Pipe size...	3½"	4"	4½"	5"	6"	7"	8"	9"	10"	12"	
45° deduct.	2⅝"	3⅛"	3⅜"	3¾"	4⅜"	5¼"	5⅞"	6⅝"	7⅜"	8¾"	
60° deduct.	.....	3⅞"	.....	4⅝"	5⅝"	6⅝"	9⅛"	.....	9⅛"	.....	

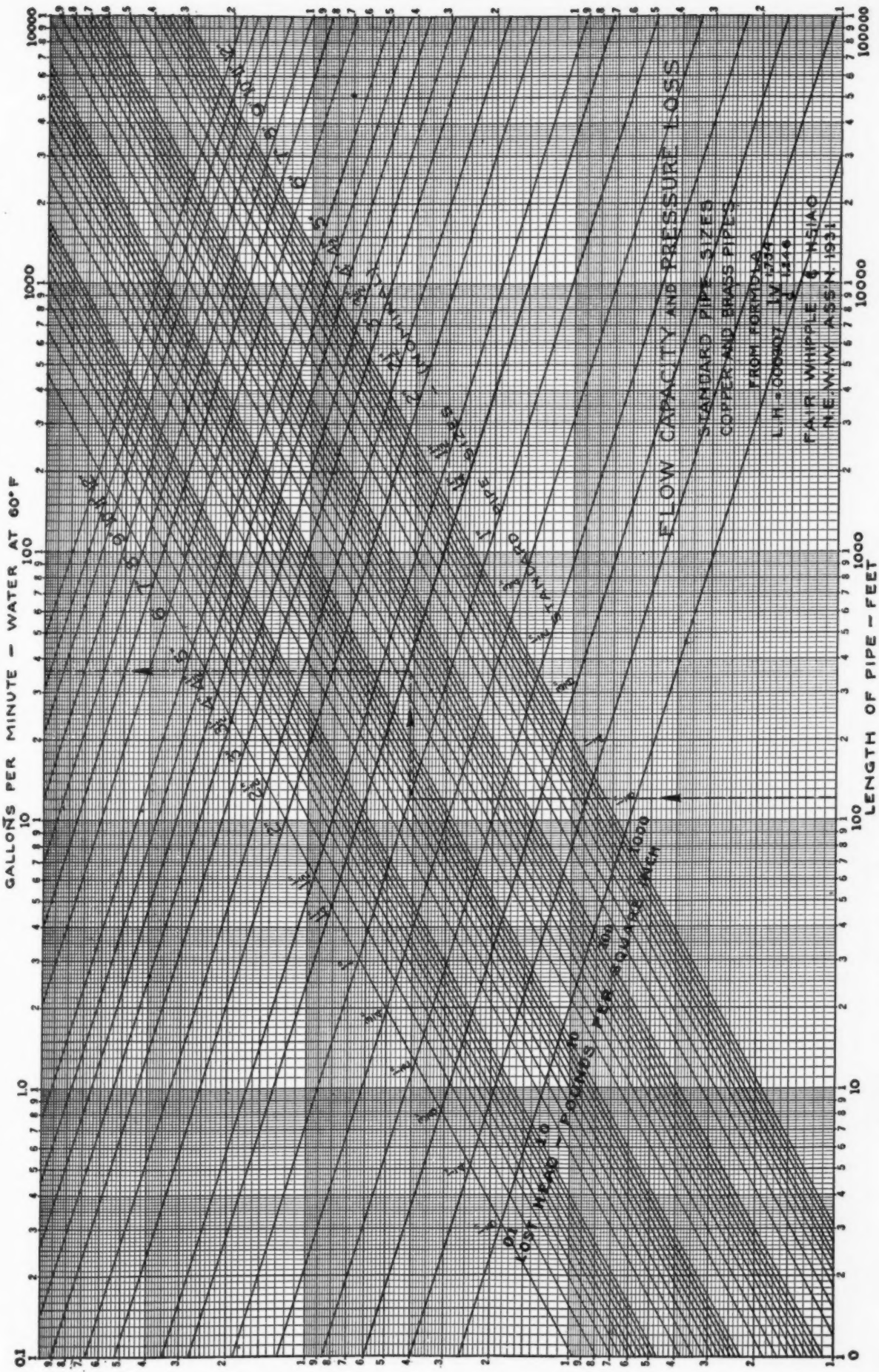
Note—The above figures represent 2xD. To find D divide by 2.

Directions

To determine the actual length of pipe, L, to use for length B, find in the table on right the amount to deduct for the desired pipe size and subtract it from length B.

For example, using the above chart, a 45° offset of width 4'-2⅛" (A) gives the length of run B as 5'-10⅞" center to center. If it is a 2" pipe then the table indicates the amount to deduct is 2". Subtracting this from 5'-10⅞" gives 5'-8⅞" as the length of pipe to cut (L).

DELIVERY CAPACITY AND PRESSURE DROP THROUGH BRASS AND COPPER PIPE



A chart developed by the Brass and Copper Research Association from formulae established at Harvard University

WATER FLOW THROUGH PIPES

A Nomograph for Solving the Hazen-Williams Formula

By DONALD R. KEITH
Civil Engineer, Division of Water, Columbus, Ohio

MANY hand-books containing hydraulic data include tables for solution of the Hazen-Williams formula for flow of water in pipes. These tables are limited in their scope and give results for "V" only. Then you either figure "Q" from "V," or find some other table which shows these values.

Even with the tables it is necessary to do a certain amount of calculation, such as interpolation, etc., for seldom does your problem fit the figures in the table.

Some books contain curve charts for the solution of this formula, but only for "V." The curve charts are usually of such small scale that they are tedious to use and will not give results close enough for practical purposes since the width of a pencil point might make a very definite difference in the value of "V."

To provide a practical device for the solution of pipe discharge problems continually confronting the Division of Water and the Sewage Department of Columbus, the accompanying chart was constructed; a chart accurate enough for all practical purposes, simple enough in its operation for an individual of average intelligence to use, and furthermore, one comprehensive enough to solve every problem that had to do with the flow of water in pipes.

The Hazen-Williams formula was selected for the reason that it seems to be more generally used and accepted than any other.

To the main formula,

$$V = CR^{0.63} S^{0.54}\ 0.001^{-0.04}$$

was added,

$$Q = AV = 4\pi R^2 V$$

in which

V = Velocity
R = Hydraulic radius
S = Slope, or loss of head
C = Coefficient.

The value of "C" depends on the material and condition of the inner surface of the pipe. (See accompanying table of "C" values.)

In laying out the R-scale, it was immediately seen that the operation of chart would be simplified by adding the pipe diameter-in-inches-scale to the opposite side of the line by the formula R = D/4.

The Q-scale was first laid off for discharge in cubic feet per second, then to simplify, a scale showing discharge in gallons per minute was added to the opposite side of line.

The scale-modulus for the R and S-scales is 8, that is to say, the logarithmic cycle for each of these scales would equal that of an 8-in. slide rule, in the original size of chart as constructed; in the C-scale the cycle is 6.84-in.; the V-scale, 3.42-in.; and the Q-scale, 1.844-in.

The S-scale is graduated in decimals of one foot to the lineal foot. In problems involving loss of head, such as one foot loss of head in a thousand lineal feet, use the point 0.001 on the S-scale, and move the decimal period three places to the right (1.00). The S-scale is used for slope, loss of head, and pressure expressed in feet.

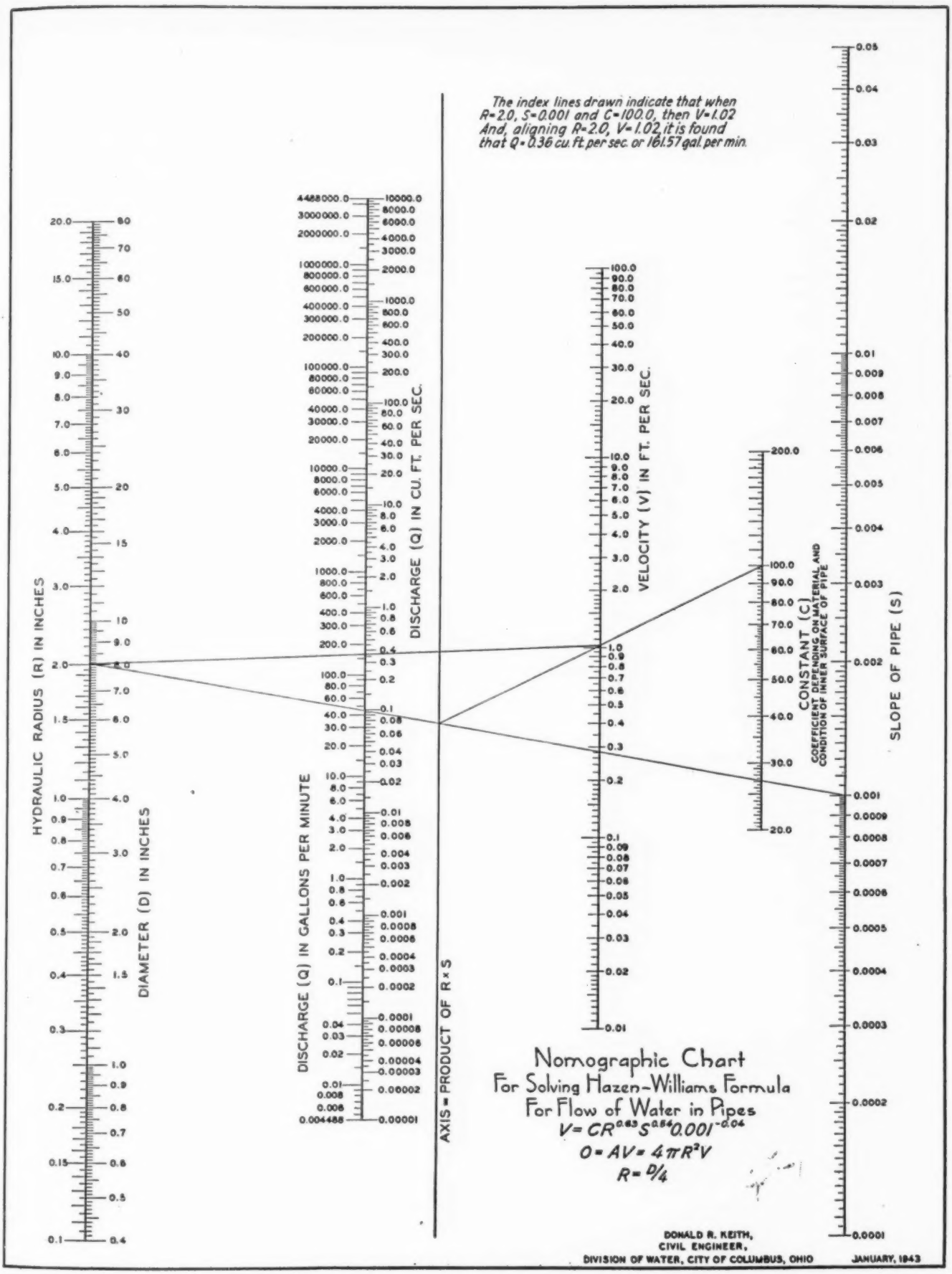
Values of "C" of course depend upon the material and condition of inner surface of pipe as noted on the accompanying chart. Tables showing values of "C" for various kinds of pipe, old and new, may be found in any hand-book of pipe data. For convenience of the user of the chart these values are listed in the following table:

"Hazen-Williams Table of Values for 'C.'"

Cast Iron Pipe, very smooth	140 to 145
" " " new, good condition	125 to 135
" " " old, good condition	100 to 125
" " " old, bad condition	60 to 100
Steel Pipe, riveted new	105 to 115
" " riveted old	90 to 105
Masonry Conduits	110 to 135
Brass, block tin, lead, glass	140 to 150

NUMBER OF TAPS THAT CAN BE TAKEN FROM PIPE OF VARIOUS SIZES

Diameter of Pipe in Inches	SIZE OF TAPS IN INCHES																							
	¾	1	1¼	1½	2	2½	3	3½	4	4½	5	6	7	8	9	10	12	14	15	16	18	20	22	24
2	11	5	3	2	1																			
2½	20	9	5	3	1	1																		
3	32	15	8	5	2	1	1																	
3½	47	22	13	8	4	2	1	1																
4	65	32	18	11	5	3	2	1	1															
4½	88	42	24	15	7	4	2	1	1	1														
5	114	55	32	20	9	5	3	2	1	1	1													
6	181	88	50	32	15	8	5	3	2	2	1	1												
7	266	129	74	47	22	13	8	5	4	3	2	1	1											
8	371	181	103	65	32	18	11	7	5	4	3	2	1	1										
9		243	139	88	42	24	15	10	7	5	4	2	1	1	1									
10		316	181	114	55	32	20	13	9	7	5	3	2	1	1	1								
12			285	181	88	50	32	21	15	11	8	5	3	2	2	1	1							
14			419	266	129	74	47	32	22	17	13	8	5	4	3	2	1	1						
15				316	154	88	55	38	27	20	15	9	6	4	3	2	1	1	1					
16				371	181	103	65	44	32	23	18	11	7	5	4	3	2	1	1	1				
18					243	139	88	59	42	32	24	15	10	7	5	4	2	1	1	1	1			
20					316	181	114	78	55	41	32	20	13	9	7	5	3	2	2	1	1	1		
22						229	145	99	70	52	40	25	17	12	9	7	4	3	2	2	1	1	1	
24						285	181	123	88	65	50	32	21	15	11	8	5	3	3	2	2	1	1	1


The index lines drawn indicate that when R=2.0, S=0.001 and C=100.0, then V=1.02 And, aligning R=2.0, V=1.02, it is found that Q=0.36 cu. ft. per sec. or 161.57 gal. per min.
HYDRAULIC RADIUS (R) IN INCHES
DIAMETER (D) IN INCHES
DISCHARGE (Q) IN GALLONS PER MINUTE
DISCHARGE (Q) IN CU. FT. PER SEC.
AXIS = PRODUCT OF R × S
VELOCITY (V) IN FT. PER SEC.
CONSTANT (C)
COEFFICIENT DEPENDING ON MATERIAL AND CONDITION OF INNER SURFACE OF PIPE
SLOPE OF PIPE (S)
Nomographic Chart
For Solving Hazen-Williams Formula
For Flow of Water in Pipes
$V = CR^{0.63} S^{0.54} 0.001^{-0.04}$
$Q = AV = 4\pi R^2 V$
$R = D/4$
DONALD R. KEITH,
CIVIL ENGINEER,
DIVISION OF WATER, CITY OF COLUMBUS, OHIO
JANUARY, 1943

Directions for using chart: Place straightedge on "D" scale at pipe diameter in inches and on "S" scale at loss of head per foot of pipe. Determine point on "Axis" line where straightedge crosses. Then place straightedge on this point on the Axis line and at Constant "C" for the pipe in question. Determine point where straightedge then crosses Velocity "V" scale. Next place straightedge on this point on "V" scale and on diameter of pipe in inches on "D" scale. Read Discharge "Q" in cfs or gpm where straightedge crosses "Q" line.

NOMOGRAPH FOR SOLVING MANNING'S FORMULA

By PAUL McH. ALBERT

Drury, McNamee and Porter, Engrs., Ann Arbor, Mich.

THE accompanying nomographic chart presents a simple, rapid, and practical means of solving Manning's Formula for the flow of water in pipes and open channels.

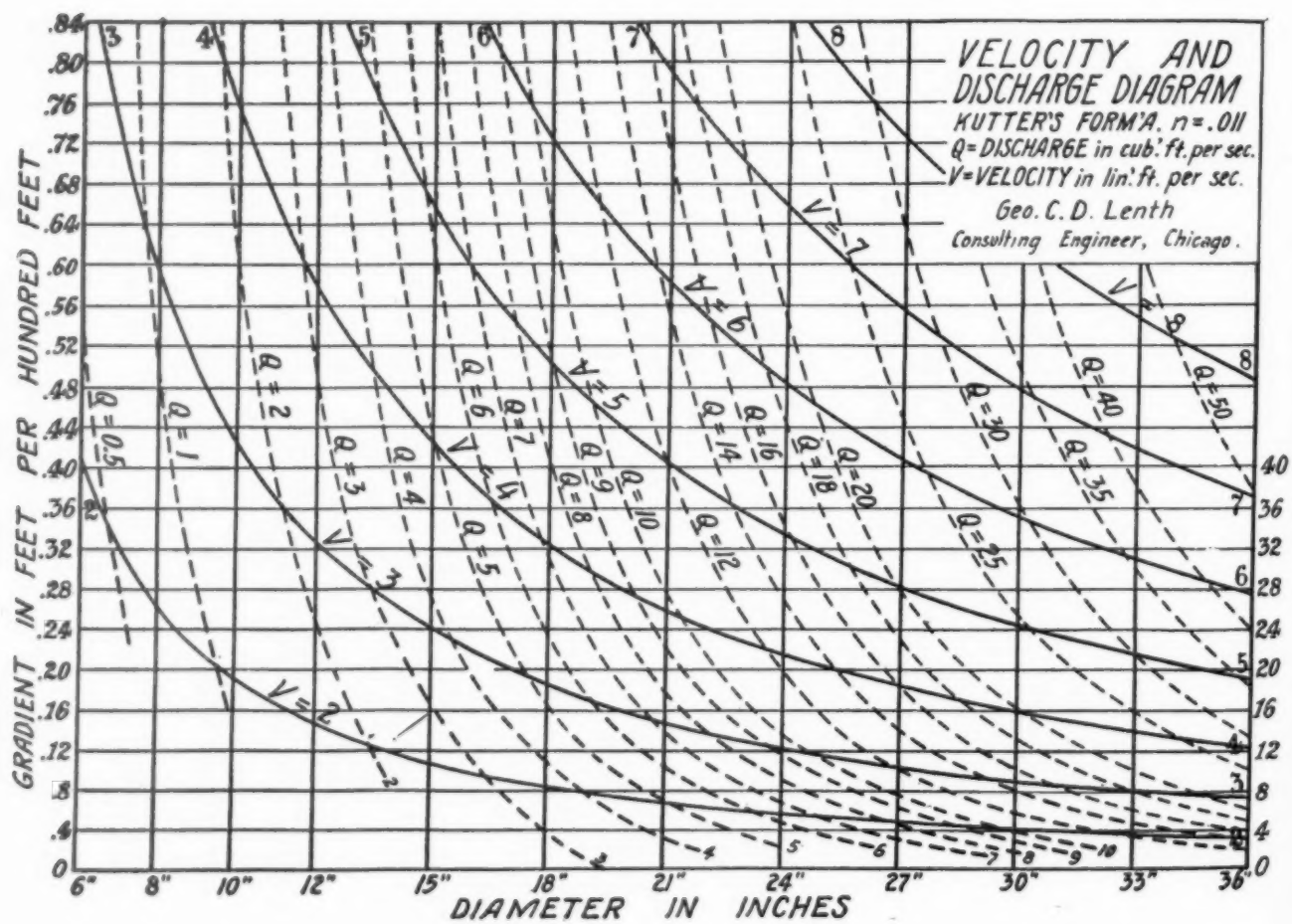
Snap analysis of problems involving this formula will be more accurate and quicker when using this nomograph inasmuch as only two or three motions are required to arrive at an answer.

If the problem entails considerable study, many different solutions may be rapidly tabulated. The effect of the coefficient "n" on the other variables as determined from the nomograph is a valuable aid for the entire range is included.

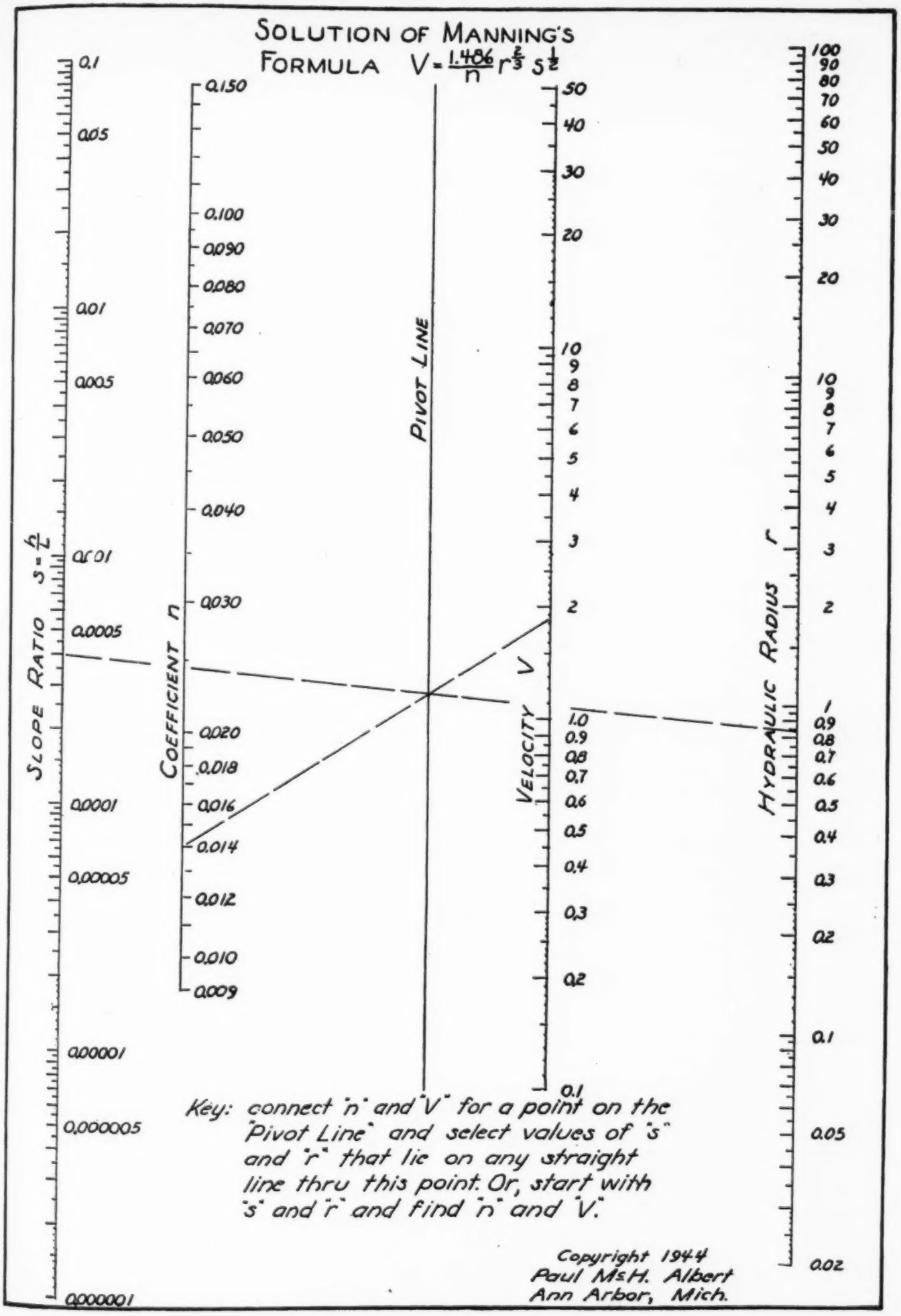
For use in checking previous designs this chart will be found to be valuable and make possible rapid checks inasmuch as it covers a wide range of all the variables. Directions for using the nomograph are given in the key at the bottom of the chart.

Velocity and Discharge Diagram for Vitrified Clay Sewer Pipe

(Courtesy, Clay Products Assn.)


VELOCITY AND DISCHARGE DIAGRAM
KUTTER'S FORM'A. n=.011
Q=DISCHARGE in cub.' ft. per sec.
V=VELOCITY in lin.' ft. per sec.
Geo. C. D. Lenth
Consulting Engineer, Chicago.
GRADIENT IN FEET PER HUNDRED FEET
DIAMETER IN INCHES
6" 8" 10" 12" 15" 18" 21" 24" 27" 30" 33" 36"

(Example: Assume 12 cu. ft. per sec. as the flow to be handled. The gradient for instance, to be a 0.2 ft. per 100 ft. run. Follow to right from gradient figure 0.20 on left until Q12 dotted line is intersected. The vertical line found at that point indicates that a 24 inch sewer is required. Also from the nearest velocity curve V.4 at the intersection it is revealed that the velocity will be 3.8 ft. at full or half full flow.)

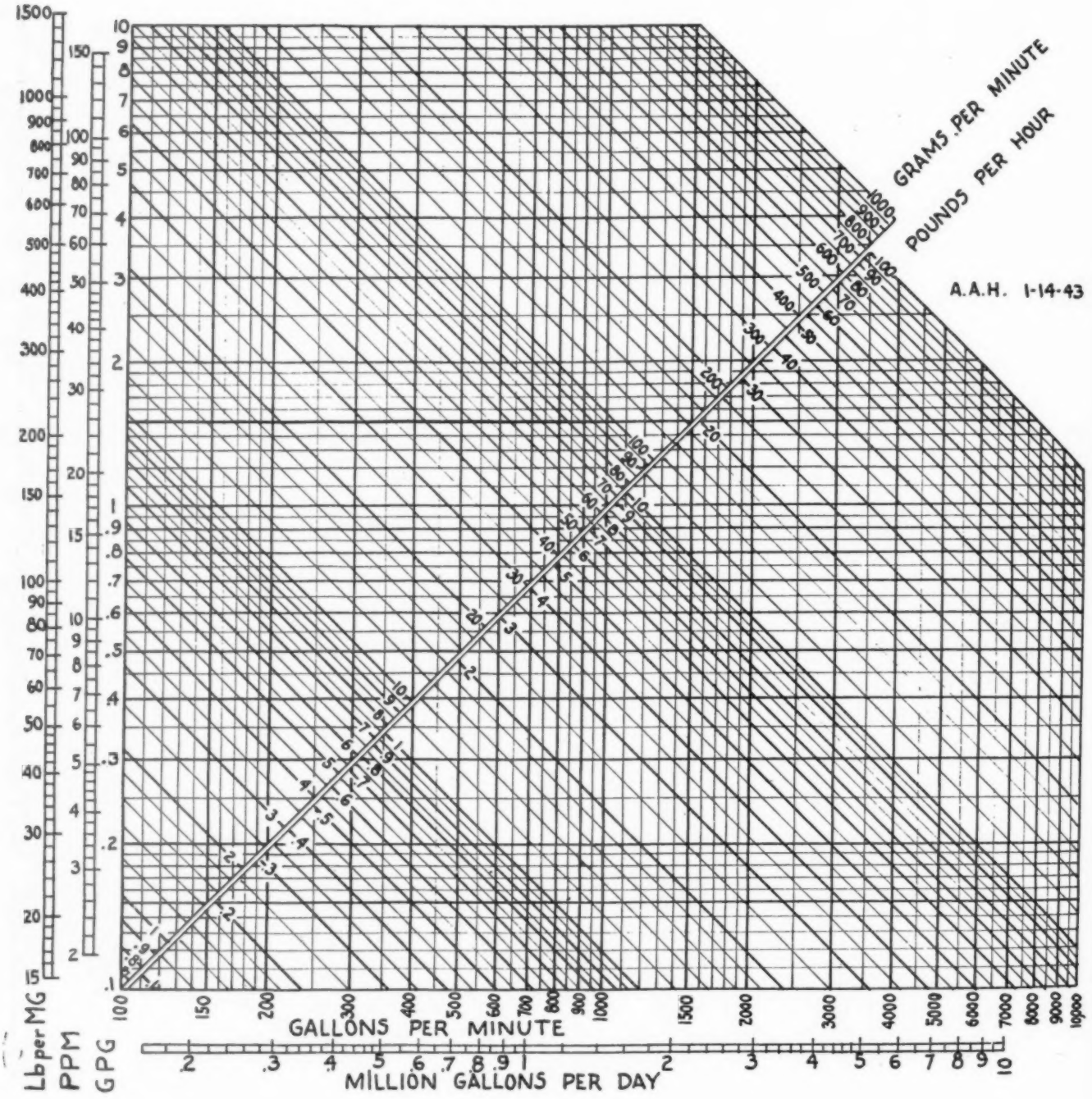


Nomograph for Solving Manning's Formula

CHECKING CHART FOR DRY CHEMICAL FEEDERS

By A. A. HIRSCH*

State Department of Education, Baton Rouge, Louisiana



The chart shown herewith offers a convenient means to check on the delivery of dry feed machines when the pumpage rate and the desired dosage is known. Rate of feed is expressed in grams per minute to enable weighing a timed discharge to check the feeder setting, and in pounds per hour to figure quickly the quantity needed in the hopper for each expected hour of operation. Chlorine dosage may be read knowing the change in platform scale readings per hour and the flow.

Parallel scales along each axis allow rapid conversion from one system of expression to another. On the vertical axis grains per gallon, parts per million and pounds per million gallons are quickly related by scanning directly across the scales. Gallons per minute and million gallons per day are similarly converted along the horizontal scale. In the plot grams per minute and pounds per hour are interchangeable across the diagonal scales. Pounds per hour is more logical from an operator's viewpoint than pounds per day. Many small plants operate only part of a day, or have a limited number of pumping steps, in which case pounds per day is merely a bookkeeping quantity.

A wide range in coverage is provided by the logarithmic divisions. High values not included on the chart may be read by using a tenth or hundredth of the quantity and applying the corresponding multiplier to the reading.

Example. How many grams per minute must be discharged from a dry feeder to give a 2 grain per gallon dosage while operating a 300 gallon per minute system?

Solution. Find the point where the 2 grain per gallon line and the 300 gallon per minute vertical intersect. This point lies between the 35 and 40 gram per minute diagonals at about 38 grams per minute. By projecting this value across the diagonal scales the hourly chemical consumption of 5.0 pounds is read.

When the answer falls in the area occupied by pounds per hour diagonals readings are converted to grams per minute by projecting in the same manner across the oblique scale.

When a metric scale is unavailable grams per minute may be obtained directly by catching the dry feeder discharge for 170 seconds and multiplying the weight in ounces by 10.

*Director, Sectional School on Water Supply and Sewerage.

FLOW AND DEPTH IN CHANNELS OR CONDUITS, CIRCULAR AND TRAPEZOIDAL

(For diagrams illustrating this article, see pages R-274, R-275, and R-276)

By PAUL McH. ALBERT

Drury, McNamee, and Porter, Engrs., Ann Arbor, Mich.

Circular Conduits

FIG 1, "Flow in Circular Channels," introduces an extremely useful solution of Manning's Formula in terms of Q, n, s, diameter and *depth*.

The designer may enter this chart with any of these variables and study the problem. It is no longer necessary to compute the full capacity first, as is currently the practice, and later use the proportional chart or tables to compute fractional flows.

The simplicity of the nomograph is due to the calculation by the writer of a factor which will be called the Circular Depth Factor. The equation shown on the chart was obtained by substituting the elements of a circular section in Manning's Formula and mathematically eliminating the terms area and hydraulic radius. Calculation of the numerical value of the last term of the equation (in brackets), the Circular Depth Factor for definite ratios of h/D from 0.10 to 1.00 produced a table of values which were graduated on the scale (4), the position of each being indicated by its h/D ratio.

Students of hydraulics will recognize the significance of the upper range of this h/D scale, which folds back on itself. The value 0.90 has no practical use and was included purely to complete the scale. It shows graphically that the flow is practically the same at 0.80 and 1.00.

Some Uses for the Nomograph

To extend the range of n, a scale of n values was placed near the bottom of the Q scale, 0.013 being placed opposite 1 cfs. It takes only a few moments to copy one cycle of the Q scale on the edge of a sheet of paper. This may then be placed on any cycle and moved up or down in accordance with the directions shown on the chart.

As the exceptionally large range of the variables includes most of the field of circular channels, only a few of the important uses for the nomograph will be noted.

The free-board in semi-circular flumes can be determined for any capacity within the size of the flume.

Prefiling of hydraulic gradients is greatly facilitated.

Storage capacities above given flows can be quickly determined.

Discharge, Area and Velocity in Partially Filled Circular Conduits

Fig. 2, "Discharge, Area and Velocity in Partially Filled Circular Sections", solves the two formulas, Q=AV and $A=xD^2$, A being common to both.

Velocity for problems in which Fig. 1 is used can be quickly determined by using the same values of Q, D and h/D. A scale for Velocity Head is placed on one side of the velocity scale (5). In this case it is not necessary to note the area, but merely use that line (3) as the pivot line.

The formula Q=AV can be used for sections that are not circular in shape—square or trapezoidal for instance.

The formula $A=xD^2$ may be also used to find the volume of horizontal circular cylinders for various depths.

Separately, the two charts offer completely independent equations, but, together, they solve Manning's Formula for all the fundamental variables encountered in circular sections. These are (Q, n, s, D, h/D, A, V) and Velocity Head.

Solution of a Typical Problem

The solution of a typical problem is carried through both charts. To find the depth of flow and velocity when n=0.013, Q=30 cfs, D=60 inches and s=1.6 ft. per 1,000 (0.16 per cent).

On Fig. 1, find h/D to be 0.372. Then depth of flow (L)=0.372×60=22.3 inches.

On Fig. 2, using the same Q, D and h/D=0.372, find the velocity to be 4.5 feet per second.

Trapezoidal Channels

Fig. 3 is for solving flow problems in trapezoidal channels. In Fig. 3, two very useful charts have been combined. On the left is the nomographic solution of the equation obtained from Manning's Formula. On the right is the logarithmic chart of d/b ratios for eight different shapes of channel, varying from vertical sides to a 4:1 slope. The two charts have a common line, the *Base Line*, which is graduated for values of the d/b ratio of vertical sides or rectangular sections.

Two straight lines intersecting at the *Pivot Line* complete the nomographic solution. The point on the *Base Line* thus obtained is then transferred horizontally to that scale of d/b ratios for the desired slope.

Sample Problem Solved

A sample solution is indicated on the chart. Here 36 cfs. is connected with 0.1 ft. per 1000 ft. and 7 ft. bottom width (b) with a point on the Base Line.

Notice that the graduation on the Base Line reads 0.76. The depth of flow (d), then, for a rectangular section 7 ft. wide would be 0.76 x 7.0 or 5.32 ft. deep. Now follow the dotted line over to the scale headed 1:1 and find there the value of d/b to be 0.5 or d equal to 3.5 ft. for a channel with side slopes of 1:1.

The d/b ratio for any other side slope can be as easily determined, or if the depth ratio in one given shape of channel is known, the depth ratio for any other given shape can be found immediately. The *bottom width* and the *flow* do not change unless the point on the Base Line is moved vertically. All values must be moved back and forth to the Base Line *horizontally*. The curved lines connect the major divisions of the various scales and are used solely to assist the eye in finding the divisions.

Construction of the Graph

Inasmuch as 0.025 is a common value of n used with earth channels, it was used for the construction of the nomograph. Other values of n may be used by shifting the Q scale up or down until the desired value of n is opposite 1.0 cfs. A convenient method of accomplishing this is to copy a cycle or two of the Q scale, in the ranges of your problem, on the edge of a sheet of paper and mark the position of 1.0 cfs. Then by placing the 1.0 cfs. mark against the desired value of n, the Q scale has been shifted and the copied scale will give the values of Q for the new value of n.

The sample equation solved on the nomograph was obtained from Manning's Formula in the following manner. Q/A was substituted for V and then A/p for r (p being the wetted perimeter). Then A and p were expressed in dimensional terms, namely: b, the bottom width; d/b, the ratio of the depth of flow to the bottom width; and the angle ø formed by the side slope and the horizontal. By transposing and combining like terms the equation was formed as shown on the chart.

The last term in the equation, in parentheses, is the factor used in constructing the scales for the side slopes. The numerical values of the factor were used to locate the position of the graduations of the ratio d/b but only the d/b ratios are given in the chart.

Comments

All the primary dimensional variables which are needed for a solution are contained in the chart. Any one, two, or more of them may not be known and often only Q is known and others must be assumed to arrive at a satisfactory answer. It matters but little which are the given values. Other values may be immediately assumed and the answer obtained in but a moment.

All other variables, as velocity, area of section, hydraulic radius, etc., can be calculated, if necessary or desired, for all the physical dimensions are known.

Most of the ordinary problems can be solved with the chart inasmuch as the ranges of the variables include a 1 ft. wide rectangular channel flowing but a few inches deep to a 20 ft. wide channel flowing 40 ft. deep.

The problems involved are those where uniform flow conditions will be satisfied. For conditions where other than uniform flow is encountered, other methods of analysis must be used.

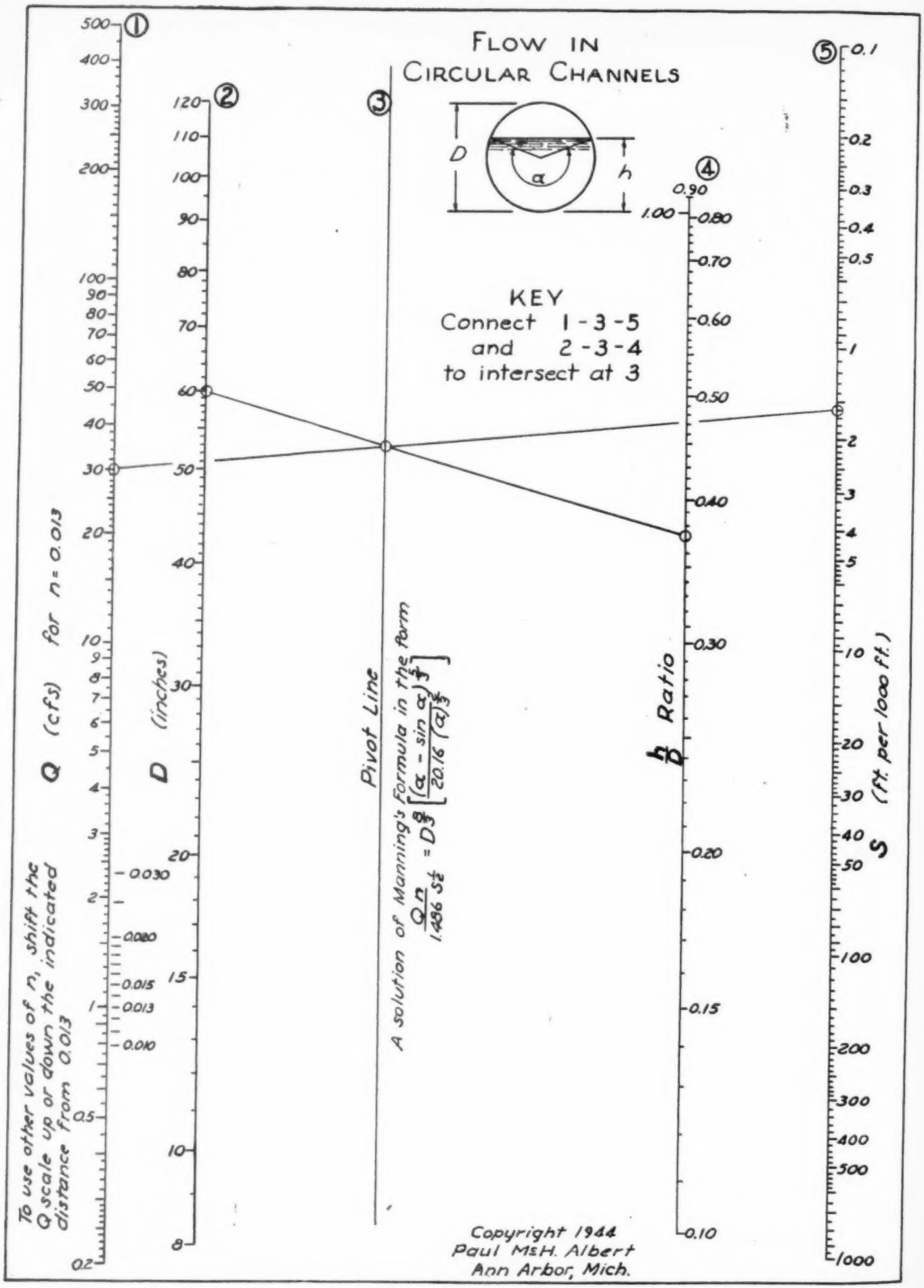

FLOW IN
CIRCULAR CHANNELS
D
h
α
KEY
Connect 1-3-5
and 2-3-4
to intersect at 3
Q (cfs) for n = 0.013
D (inches)
Pivot Line
h/D Ratio
S (Ft. per 1000 ft.)
To use other values of n, shift the Q scale up or down the indicated distance from 0.013
A solution of Manning's formula in the form
Q n / 1.486 S^½ = D^8/3 [(α - sin α) / 20.16 (α)^5/3]
Copyright 1944
Paul McH. Albert
Ann Arbor, Mich.

Fig. 1.—Flow in Circular Channels

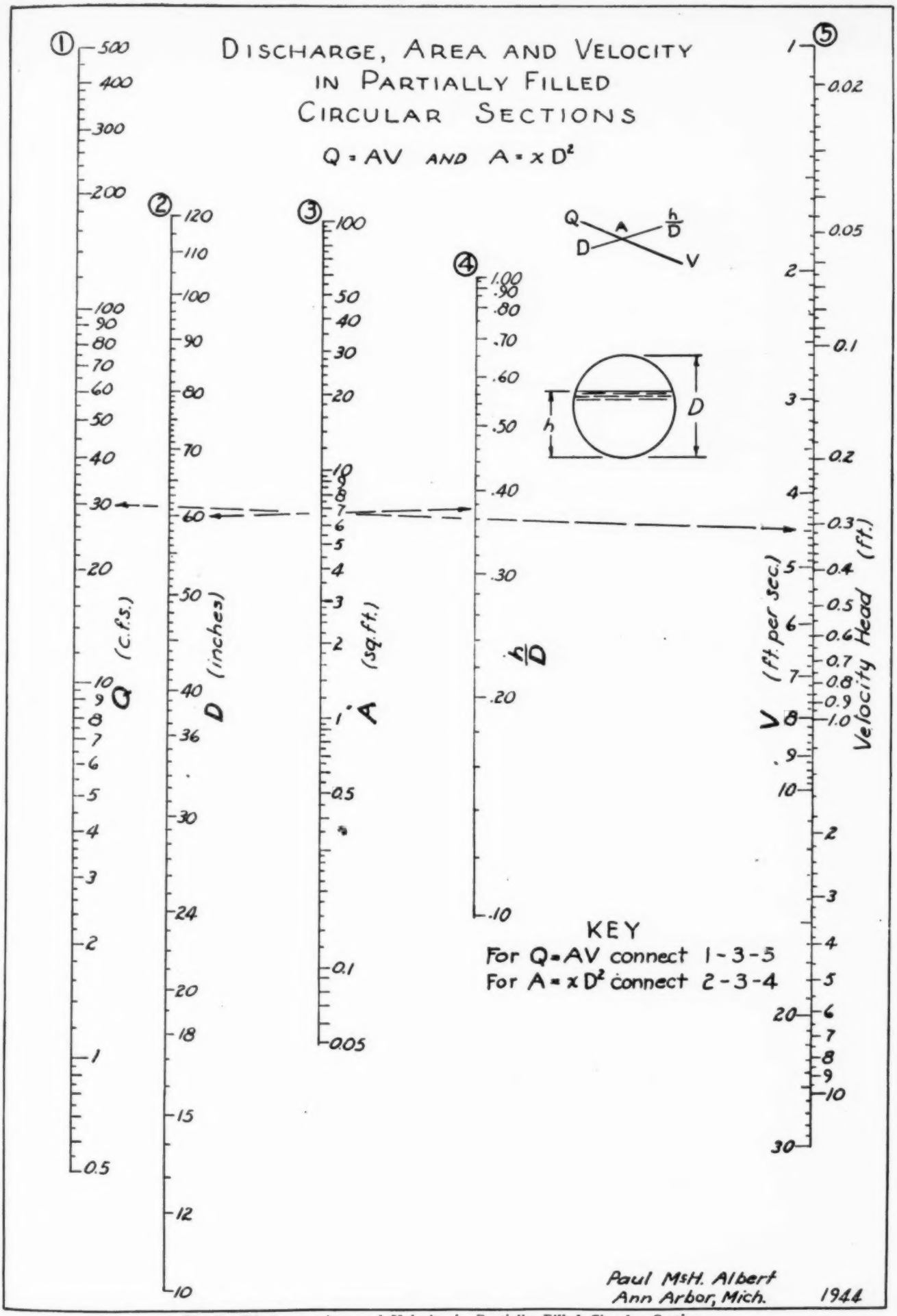
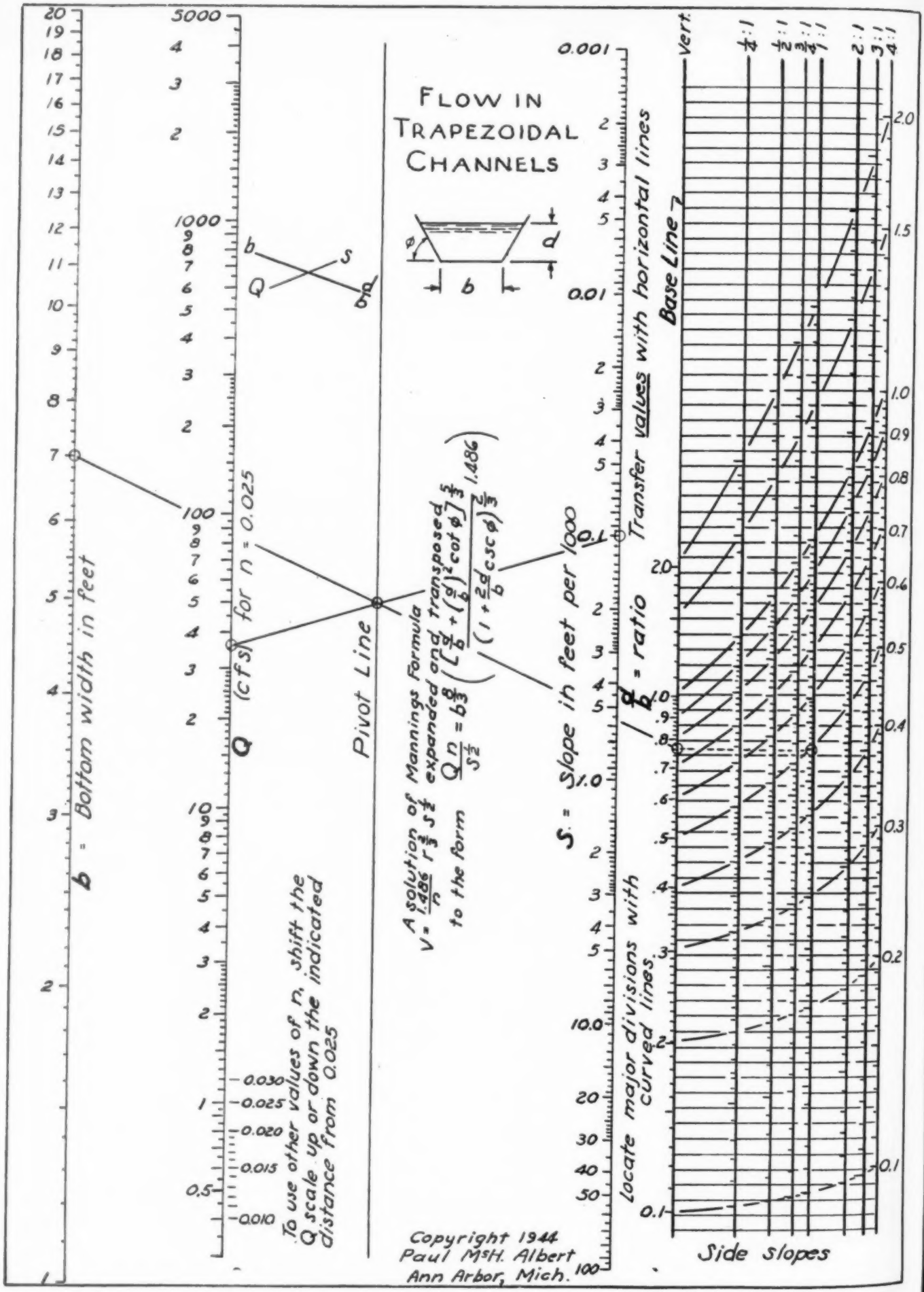


Fig. 2.—Discharge, Area and Velocity in Partially Filled Circular Sections


FLOW IN TRAPEZOIDAL CHANNELS
b = Bottom width in feet
Q (cfs) for n = 0.025
S = Slope in feet per 1000
d/b = ratio
Transfer values with horizontal lines
Base Line
Pivot Line
Locate major divisions with curved lines.
Side slopes
Vert.
4:1
3:1
2:1
1:1
To use other values of n, shift the Q scale up or down the indicated distance from 0.025
0.030
0.025
0.020
0.015
0.010
A solution of Mannings Formula
expanded and transposed
to the form
Copyright 1944
Paul MSH. Albert
Ann Arbor, Mich.

CONSOLIDATED REFERENCE AND DATA INDEX 1941 through 1948

All subjects in this and previous issues are listed with the respective key,—('41), ('42), ('43), ('44), ('45), ('46), ('47), or ('48)

A

A-B-C of Depreciation ('47) Mag. 237
Accounted for Water and Unbilled Revenue ('48) R-100
Accounting Practices, Municipal Depreciation ('45) R-15
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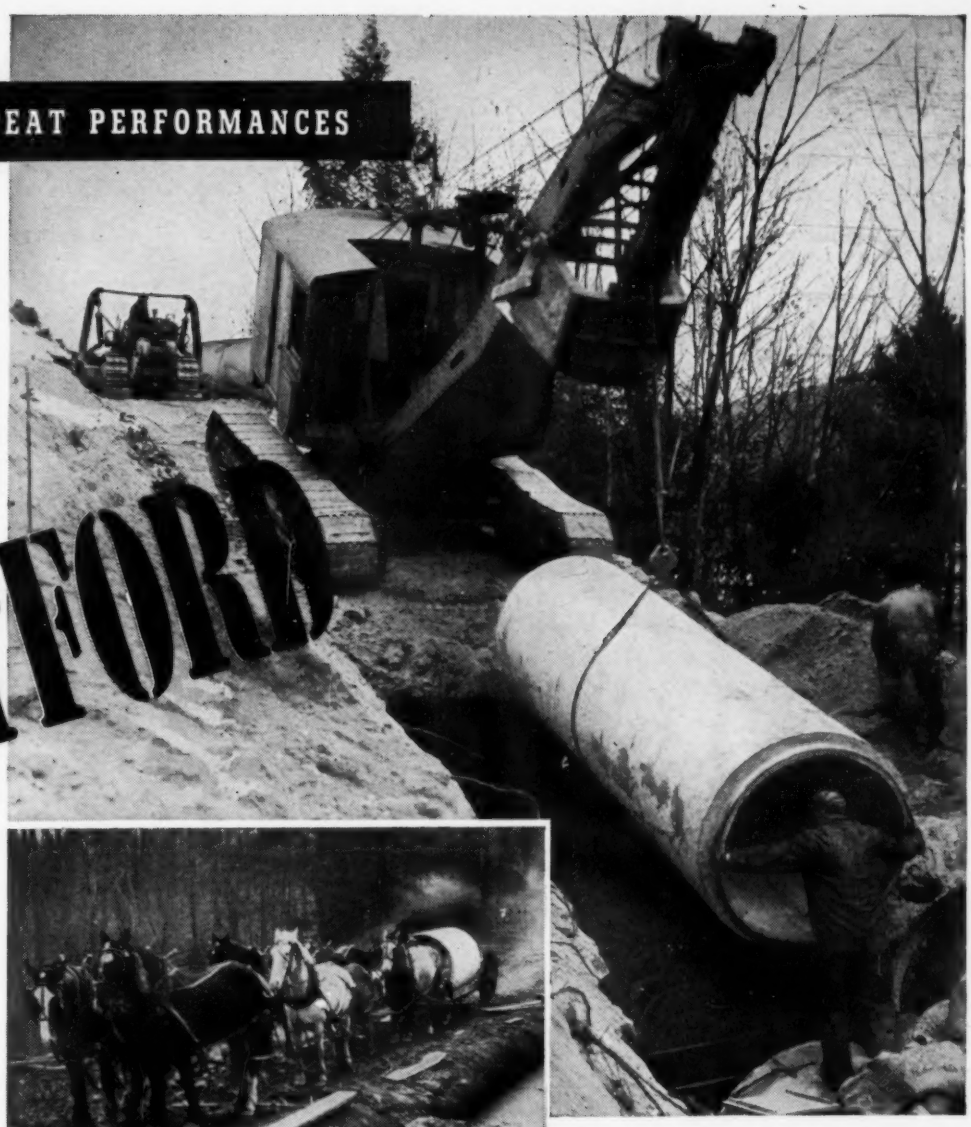
LOCK JOINT REPEAT PERFORMANCES

HARTFORD

9 JOBS SINCE 1918



1940 – Above: Modern installation of 48" concrete cylinder pipe.

1918 – Left: Delivery of 48" pipe by six horse team.

For almost 30 years the increasing demand upon the water supply system of Hartford has been met with Lock Joint pressure pipe lines. Since 1918 nine separate contracts for supplementing the City's existing water supply system have been awarded to Lock Joint Pipe Company. By specifying Lock Joint Reinforced Concrete Pressure Pipe, Hartford, the insurance capital, has assured itself of water supply lines of unfailing service whose estimated life expectancy is better than 100 years!

Repeat orders from Hartford and many other progressive American cities are convincing evidence of the permanent high quality and complete reliability of Lock Joint Reinforced Concrete Pressure Pipe.

SCOPE OF SERVICES—*Lock Joint Pipe Company specializes in the manufacture and installation of Reinforced Concrete Pressure Pipe for Water Supply and Distribution Mains in a wide range of diameters as well as Concrete Pipe of all types for Sanitary Sewers, Storm Drains, Culverts and Subaqueous Lines.*

For future projects, large or small, specify Lock Joint Pipe.

A Hazen-Williams Hydraulic Slide Rule will gladly be sent upon request with our compliments.

★

LOCK JOINT PIPE COMPANY

Established 1905

P. O. Box 269, East Orange, N. J.

Pressure Pipe Plant: Wharton, N. J. • Sewer Plant: Kenilworth, N. J.

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A report by the sanitary engineer and chemist of the New Haven Water Company shows that when the Break Point Process was applied virtually all gas-formers were eliminated.

However, bacteriological improvement was not the only gain. According to an article in American City[1] summarizing the report:

"****In the winter and spring***several problems arose that made the company appreciate the practice of using breakpoint chlorination. When the***reservoir was covered with ice, *Asterionella* developed to the extent of 1,300 standard areal units per ml., producing a fishy odor in the untreated water.

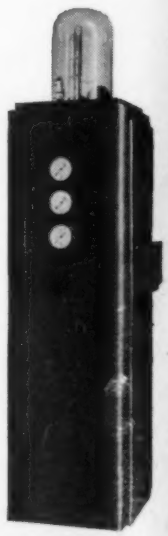
After treatment, however, not a trace of odor was noticeable."

In addition, although its water was well within U.S.P.H. standards,—

"****the***Company***has endeavored to produce as good a water as water works practice would allow; and they have been convinced that the use of breakpoint chlorination throughout the system is well worthwhile."

New Haven's experience is typical. Regardless of plant size, the Break Point Process can help you produce the kind of pure, palatable water consumers like. To find out how these and additional benefits such as—longer filter runs—iron and manganese removal—and color removal, can be obtained, check with your nearest W & T Representative.

[1]Issue of November 1946



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